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HANDBOOK No. 10

STRUCTURAL STEEL

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H. C. Holliday
Balmain.

JULY, 1906.

HANDBOOK No. 10.

STRUCTURAL STEEL

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INTRODUCTION.

THE value of any work of reference is greatly enhanced if the possessor take the trouble to acquire in advance a precise estimate of its scope. For this reason, the reader is strongly urged to take the first opportunity of reading this book through, even if he have no immediate occasion for putting it to practical use.

Brief mention should here be made of one or two characteristic features of this handbook.

(1) From time to time a steel user has occasion to design a new steel shape, either because no existing section is suited to the particular purpose in view, or because, having a large quantity to purchase, he takes the opportunity of indulging his own idiosyncrasies. The result is that the manufacturer who secures the order in question turns the special rolls required, and, having executed the order, frequently includes the new section in his catalogue in the hope of attracting further orders. It need hardly be said that, as a rule, little or no demand is created, and ultimately the rolls are scrapped.

It follows that the draughtsman who uses a single manufacturer's catalogue runs a risk of specifying sections of which delivery either cannot be obtained at all or cannot be obtained without paying a high premium to have rolls mounted specially.

The present handbook is free from this objection:—No shapes or sections have been included, without a special note to that effect, which are not rolled regularly and at short intervals in response to a continuous demand.

Besides entering a caveat against a few sections which are almost unobtainable at present, the various sections have been classified as precisely as circumstances permit. Generally speaking, the classification should be interpreted as follows:—

(*a*) Those accustomed to buy from retail girder or stock merchants should preferably confine themselves to sizes stated to be stocked or freely obtainable in any quantities.

(*b*) Buyers requiring delivery from mills for orders of 5 to 10 tons of a size may specify sizes stated to be "frequently rolled" with as much confidence as sizes stated to be stocked.

(*c*) Some of the standard sections are practically unobtainable unless ordered in lots of about 50 tons of a size. These sections the reader is recommended to avoid, in the foot-notes to the tables. A few of them may come into regular use in course of time; others are not unlikely to be removed from the official list of standard sizes when revised from time to time.

(*d*) Sections which do not come in any of the above categories can generally be obtained promptly from wholesale iron merchants, but may cause trouble if ordered from a constructional engineer, as part of an order for steelwork, or from a manufacturer without specific assurance as to delivery.

(2) What has been stated with regard to manufacturers' catalogues, applies similarly to most of those issued by retail girder merchants and constructional engineers. Such catalogues would obviously defeat the object with which they are issued, if they did not tend to restrict the steel

user to a single supplier. This handbook is designed for the use of those who object to any such restriction and desire open competition among their suppliers for any steelwork they may require. The draughtsman who selects rolled steel shapes from this book can invite tenders from any number of respectable constructional engineers or girder merchants with the certainty that all can obtain their raw material promptly and at lowest wholesale prices.

(3) This handbook will be used by many different classes of readers whose technical qualifications vary considerably. This will explain why the whole of the text has been worded in simple non-technical language; while, on the other hand, the book includes some information intended chiefly for the benefit of the civil engineer, which a few readers may deem superfluous or confusing.

(4) The tables of safe loads etc. are not designed for the benefit of those who are unable to make their own calculations, but as time-saving appliances. Their advantage as such is to be measured, not so much by the actual saving in time, as by the advantages to the draughtsman of being freed from the distracting effect of having to make calculations, however simple, while engaged in design. It is equally distracting to have to search through an ill-arranged collection of tables. For this reason, exceptional care has been devoted to ensure ease and rapidity of reference. It is needless to mention the many original devices which have been employed to this end, as they will be evident to every one who peruses this book before using it, as already recommended.

(5) Exceptional care has been taken to avoid errors in calculation. Every figure has been carefully verified, and the various tables calculated at least twice by independent formulæ etc. The proofs have been revised as scrupulously as the original MS. This book is free from "padding," such as matter extracted without verification from mathematical tables and familiar text-books.

Owing to the extreme care exercised in the compilation of this book, it seems improbable that any material error can have escaped correction.

(6) The original calculations for this book were generally made to three decimal places or more, but the results are only published herein to as many decimal places as are required for practical purposes. The "Properties" of British Standard Sections, which have been published by the Engineering Standards Committee to three places of decimals, have been pruned down in this book in like manner. Apart from this modification, they are copied, by permission, from the Engineering Standards Committee's official List.*

* "Properties of British Standard Sections," July, 1904, published by Crosby Lockwood and Son, London.

PART I.

BROAD FLANGE BEAMS.

NOTES ON PART I.

SECTION DRAWINGS, PROPERTIES AND SAFE LOADS.

GENERAL EXPLANATION.

There are 23 existing sizes of Broad Flange Beams, ranging in depth from 7 to 30 inches. Detailed particulars of these Beams are given on pages 24 to 115. Four consecutive pages are devoted to each size of Beam, and the various sizes are arranged and indexed in order of depth. The data given for each size are as follows:—

First page :—Section drawing.
Second „ :—Properties and safe loads.
Third „ :—Drawings of standard connections.
Fourth „ :—Details as to sizes, weights etc. of the standard connections illustrated on the third page.

These tables and drawings have been made as self-explanatory as space admits, but the following general explanation should be read before putting this book to practical use. If this is done, it will not often be necessary to refer to different parts of the book in order to understand the tables and drawings.

SECTION DRAWINGS.

The section drawings are reproduced to scale and are fully dimensioned.

The web and flange thicknesses are given in decimals and also in fractions of an inch (nearest 64th). All dimensions are given in metric equivalents also, these being so arranged as to be readily distinguishable from the rest. When one drawing has been used, the rest will be equally familiar, as the dimensions are set out in uniform fashion throughout.

Various improvements will be noticed in the design of Broad Flange Beams as compared with other standard H sections, owing to the improved type of rolling mill in which Broad Flange Beams are produced (further explained on page 224).

PROPERTIES AND SAFE LOADS.

The information given on pages 25 etc. is as complete as space permits, but some useful data have necessarily been excluded, so that the reader should take note of the supplementary data tabulated in Part II. (*vide* synopsis on page 116). The principal tabulated properties may be defined briefly as follows:—

MOMENTS OF INERTIA.

If a section is imagined to be divided up into an infinite number of small parts, the sum of the areas of these parts each multiplied by the square of its distance from a given straight line or axis, is called the "Moment of Inertia" of the section about the given axis. In the case of H beams, the moments of inertia of practical interest are those about the axes marked XX and YY on the various drawings. These are the greatest and least moments of inertia respectively, and are often so described.

SECTION MODULI.

The section moduli of H sections are calculated as follows:—

Greatest Section Modulus (XX) = M. of Inertia (XX) $\div \frac{1}{2}$ depth.
Least „ „ (YY) = „ „ (YY) $\div \frac{1}{2}$ flange width.

NOTES ON PART I.

WEIGHTS.

MOMENTS OF RESISTANCE.

The "Moments of Resistance" tabulated in this book correspond to a "working" or flange stress of $7\frac{1}{2}$ tons per square inch. That is:—

Tabulated "Moment of Resistance" = $7\frac{1}{2} \times$ Section Modulus (xx).

[N.B.—The term "Moment of Resistance" has unfortunately become ambiguous, being often used as if synonymous with the term "Section Modulus."]

When a beam is loaded irregularly, the simplest way to ascertain what size of beam is required, is to calculate the maximum bending-moment due to the load. (The formulæ for some simple cases are given on page 124.)

It then only remains to select a beam of which the "Greatest Moment of Resistance" is equal to the ascertained bending-moment (ton-inches).*

RADII OF GYRATION.

To calculate the radii of gyration, divide the Moment of Inertia by the sectional area (square inches), and take the square root of the result.†

NETT WEIGHTS.

The calculated weights (in cwts.) of B.F. Beams of various lengths are given on pages 25 etc. in the table of safe loads on stanchions. For example, on referring to the particulars for 10″ × 10″ section, the weight under 19 feet is given as 9·40 cwts., this being the calculated weight of a plain 19 feet length.

Weights can also be roughly estimated by referring to the "feet per ton" given in the "Properties" at the top of the page; for example, in the case of 10″ × 10″ section, it is stated that 40 feet go to the ton, so that obviously a 19 feet length weighs rather less than $\frac{1}{2}$ ton.

GROSS WEIGHTS.

The weight given on pages 25 etc. for "end-fastenings to a girder" represents the weight of one pair of standard web cleats and upper flange cleat.

For estimating the gross weight of a stanchion, the weight of the standard base and the weight of the heavy pattern of cap shown in the drawings are stated.

If the nett weights of the main girders and stanchions have been ascertained (by "nett weights" are meant the weights of the plain rolled steel beams without fittings), the gross weight of the steelwork in a building of several stories can be estimated roughly as follows:—For every 100 tons nett weight of main girders and stanchions, there will be about 10 to 12 tons in connections, and about 4 to 6 tons in "binders" (*i.e.* light H sections used as tie-beams to the stanchions).

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* If the moments of resistance are not tabulated, divide the ascertained bending-moment (ton-inches) by the desired working stress (tons per square inch).

The result gives the required "Section Modulus (xx)," and by referring to the tables of properties, a suitable size of beam can be determined without further calculation.

† If the greatest and least radii of gyration are known, any other radius can be ascertained graphically by describing an ellipse of which the major and minor axes are equal to twice the greatest and least radii of gyration respectively. This is the "ellipse of inertia."

NOTES ON PART I.

DRAWINGS OF STANDARD CONNECTIONS.

CONNECTIONS TO BROAD FLANGE BEAMS.

The great saving in the weight and cost of steelwork attending the use of Broad Flange Beams is a striking advantage. But experienced engineers are inclined to attach even greater importance to the remarkable facilities which they offer for first-class fastenings of the strongest and simplest possible character.

The drawings presented in this book are intended chiefly as examples and suggestions.

To standardise connections etc. in such a way as to dispense with the necessity of technical knowledge and experience is a manifest impossibility and is not attempted.

Most of the sheets of drawings comprise the following :—

Fig. 1. End elevation of end-fastenings for a girder ("Web and Flange Cleats").

Fig. 2. Side elevation do. do. do.

Fig. 3. Plan of web cleats shown in Figs. 1 and 2.

Fig. 4. A pair of ordinary fishplates as used to join the ends of two girders resting on the same stanchion.

Fig. 5. End elevation of stanchion, showing standard cap ("light" pattern) and base.

Fig. 6. End elevation, showing an alternative "heavy" pattern of cap.

Fig. 7. Side elevation of stanchion, showing standard cap ("light" pattern) and base.

Fig. 8. Plan of stanchion base.

These are dealt with in order, but it will simplify the explanation if some of the principles which should govern the design of connections in general, and some characteristic features of the drawings as a whole, are first enumerated.

CONNECTIONS IN GENERAL.

(1) The strength of connections must obviously be proportionate to the vertical loads.

(2) Connections should also be proportioned and arranged with a view to providing the best practicable end-fixing and lateral stability for stanchions etc. and rigidity to the structure as a whole. The latter feature depends mainly on the strength and rigidity of the joints. It is false economy in any case to neglect fastenings, because, in proportion to their strength, they stiffen the girders and stanchions, adding appreciably to their value as such. If cost must be kept low, relatively light members well fastened make a better and cheaper structure than heavier members badly fastened.

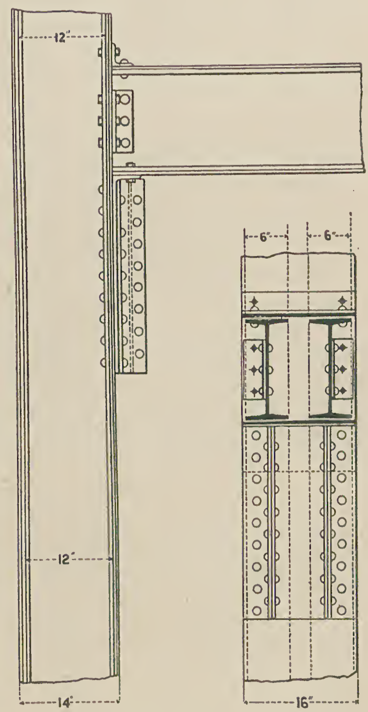
(3) Connections should be of the simplest character both in order to keep down expense in the engineering shops and with a view to the structure being assembled at site quickly and cheaply.

These primary conditions are fully complied with in the case of well-designed structures of Broad Flange Beams. Narrow-flanged material, whether in the form of ordinary rolled steel joists or riveted girders and stanchions, presents many difficulties.

The first condition is usually complied with; that is, the connections to compound girders and ordinary joists can be and naturally are

proportioned to the vertical loads. But the second condition is often ignored as it would be difficult to observe it. The difference is clearly shown in the illustration on page 23. The wide cleats riveted to the flanges of the girders contribute very effectively to the end-fixing of the stanchions, and to the general stability of the structure. If narrow-flanged joists were substituted, it is obvious that the end-fixing would be of much less value as regards lateral stability, not only on account of the impossibility of making good flange connections, but also because the resistance of narrow-flanged girders to side deflection or twist is relatively insignificant as compared with that of Broad Flange Beams.

Some types of compound girders are better than ordinary joists in this respect, but it is usual to secure the upper flanges by a couple of bolts only, as the interior flanges of double-webbed girders are practically inaccessible; moreover, the bolts are necessarily of small diameter, owing to the narrowness of the flanges. Consequently, the fastenings are totally disproportionate to the lateral stiffness of the girder and only a small portion of its strength is available for lateral stability.* (See foot-note, page 12.)



The great value of connections to Broad Flange Beams lies in the fact that rivets and bolts of large diameter can be placed just where they are most required, so that the sum of their moments of resistance about each axis of possible rotation is considerably greater (usually at least 50 % greater) than that of the best practicable connections to ordinary joists or riveted sections.

As regards cheapness and simplicity, the experienced user need only compare the drawings of typical joints on pages 22 and 23 with similar joints, designed to carry the same loads, as required for built-girders and stanchions. The fastenings to Broad Flange Beams are of the simplest possible character, consisting usually of simple angle cleats. Connections of compound girders to stanchions generally comprise gusset plates, vertical angles and packing plates ("fillers") *in addition* to web and flange cleats.

A typical joint to built-members is illustrated in the accompanying sketch. It will be observed that, in order merely to provide the requisite number of bolts and rivets in vertical shear, somewhat elaborate gusseting has to be resorted to. No adequate provision is made for lateral stability as this would entail encroaching unduly on the inner space of a building. If Broad Flange Beams were used in place of these built-sections, the depth of

the fastenings could be reduced by 50 % or more, as they admit of double rows of bolts or rivets in each flange. The bolts and rivets in connections to built-members are necessarily of relatively small diameter, consequently a greater number is required than in similar connections to Broad Flange Beams. Three $\frac{7}{8}''$ rivets will do the work of four $\frac{3}{4}''$ rivets.

As regards convenience in erection, all properly designed structures composed of Broad Flange Beams can be assembled by unskilled labour, easily and rapidly. Every nut and bolt-head is accessible to ordinary tools. In fact, simple structures composed of plain rolled steel beams economise time and cost in every department—in design, in the shops, and in erection.

GENERAL CHARACTERISTICS OF THE STANDARD CONNECTIONS.

(1) The various "standard" connections are not only fully equal to the vertical loads to which they are proportioned, but are also such as to contribute materially to the strength of the members joined and to the general stability of any structure in which Broad Flange Beams are employed.

(2) The diameters of bolts and rivets and thicknesses of plates and angles are proportioned as usual to the thicknesses of the parts joined. As Broad Flange Beams have thicker webs and flanges than narrow-flanged joists and other sectional material ordinarily employed in making riveted girders etc., it naturally follows that in connections to Broad Flange Beams the rivets used are larger, and proportionately less in number, than would generally be employed for built-members carrying similar loads.

Similarly, the individual plates and angles are somewhat thicker and heavier than usual in built-sections of corresponding dimensions.

The connections are further simplified by the fact that most of the sections permit the use of a double row of rivets or bolts through both sides of the flanges.

(3) In consequence of these features, the most suitable connections to Broad Flange Beams differ from ordinary connections chiefly in being made of a few strong and simple instead of a multiplicity of lighter and more complex parts. Each particular fastening is lighter as a whole, simpler, more effective and much less costly than the best practicable fastenings to riveted girders and stanchions of similar dimensions.

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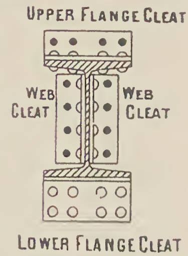
* Those having a limited knowledge of engineering gained solely from building practice are apt to imagine that considerations of lateral stability etc. are purely academic and need not be seriously considered. This is not altogether unnatural, because the building regulations in this country make it difficult at present to realise the advantages of properly designed steel-framed buildings. But, even in the case of a building surrounded by relatively massive walls, it is questionable whether the lateral stability of the interior steelwork can be safely ignored. The accidental overstraining of an individual member during erection may result in the total collapse of an ill-jointed structure, as in the case of the Darlington Hotel. The partial destruction of a building by fire may also cause the general stability of the steelwork to be severely tested. It might not unreasonably be urged that these and similar contingencies are too remote for practical consideration; but, as far as it is possible to provide against them by reasonable attention to fastenings, without materially increasing cost, it is obviously rational to do so. From a purely commercial standpoint also, the arguments are entirely in favour of well-fastened steelwork. The purchaser of a building may possibly be content with a very bare margin of strength if attended by an adequate saving in cost. But it would not be considered business-like to accept ill-jointed steelwork, if it were realised that the same sum of money if invested in somewhat lighter but better-proportioned and strongly fastened steelwork would mean a considerably increased margin of safety. [*See also notes on page* 10.]

DRAWINGS OF STANDARD CONNECTIONS.—Continued.

WEB AND FLANGE CLEATS (Figs. 1 to 3).

The web and flange cleats are designed with a view to joints of the character illustrated on pages 22 and 23, where each girder is attached to the stanchions by two pairs of angle cleats.



The distributed loads to which the end-fastenings are proportioned (and the spans corresponding to these loads at 7½ tons working stress) are stated in the notes at the foot of the drawings.

The strength of the "web cleats by themselves" is represented by the value of the connection bolts in single shear (here taken as 4 tons per square inch).

The cleats are riveted to the web by a double row of rivets in order to obtain a good hold on the web.

In fixing the length (or depth) of the web cleats, it is assumed that the beam will be bolted to an angle bracket, and room has accordingly been allowed for nuts or bolt-heads. If there are no flange connections, the web cleats may be made of a length equal to the clear depth of the web as indicated on the section drawing.*

LOWER FLANGE CLEAT (Figs. 1 and 2).

This is an "angle bracket" intended to be riveted to a stanchion or to the web of a deeper girder to carry the end of the beam in question.

The notes at the foot of the various drawings state the "distributed loads" to which the "combined Web and Flange Cleats are suited."

The stated safe loads assume a shearing value of 4 tons per square inch for the rivets and only 3 tons per square inch for the bolts in this case, because less of them come in shear when the girder rests on an angle bracket.

The bracket is shown in dotted lines, because it will usually be riveted to some other member of the structure in which the beam is employed. This cleat will be set back from the edge of the girder, as shown in the drawings, in cases where the beam is provided with a landing on the edge of a stanchion as in the typical joint illustrated on page 22.

If the "lower flange cleat" forms part of the end-fixing of a stanchion as in the typical joint on page 22, it is desirable to allow a greater margin of strength than in cases where the sole function of the bracket is to carry a part of the load of the girder. [*See notes on* "*Stanchion Caps.*"]

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* In cases of the above kind, the stresses in the connection bolts are generally assumed to be simple shearing stresses. Actually, the inevitable deflection of the beam puts the upper bolts in tension, and until these have been strained to a certain extent, the lower bolts cannot come into play.

The required number of rivets in the web cleats is usually determined by their value in double shear (or by their bearing value on the web, whichever is the smaller).

From this point of view, the drawings appear to show more rivets than are required to balance the strength of the connection bolts.

A slight disproportion as regards the vertical loads is inevitable, as a good hold of the web is desirable for lateral rigidity. It is not, in any case, correct to assume that the shear on the rivets is exactly equal to the end-reaction (*i.e.* the load on the connection bolts). It is the end-reaction multiplied by the leverage with which it acts; that is, by the distance of the centre of gravity of the group of rivets from a line drawn through the centres of the bearing surfaces of the connection bolts. This product is the moment of the load. The resistance offered by the rivets is measured by the moment of resistance of the group about an axis drawn perpendicular to the surface of the web through the centre of gravity of the group.

DRAWINGS OF STANDARD CONNECTIONS.—Continued.

UPPER FLANGE CLEAT (Figs. 1 and 2).

This will be required when the girder is attached to the foot of an upper stanchion as on page 22. Its principal function is to contribute to the fixing of the stanchion. Incidentally, it stiffens the beam also. The bolts in the upper flange cleat should not be estimated to carry any part of the load.

If the depth of the beam exceeds $1\frac{1}{2}$ times the least width (*i.e.* the flange width) of the stanchion, the upper flange cleat may be omitted.

BOLT-HOLES IN UPPER FLANGE CLEAT.

The arrangement of the bolt-holes shown in the "upper flange cleats" will naturally have to be varied in many cases. If, for example, a beam with 12″ flanges is attached to one flange of a 10″ × 10″ stanchion, the bolt-holes through the angle should naturally be pitched at the same centres as the bolt-holes in the standard flange cleats for 10″ × 10″ section; similarly $\frac{3}{4}''$ bolts should be used instead of $\frac{7}{8}''$ bolts. This point is made clear in the following sketches.

FIG. A.

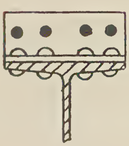


FIG. B.

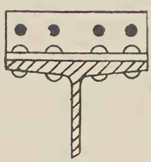
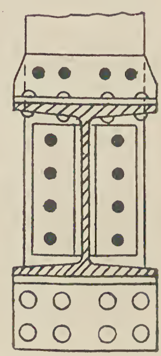


Fig. A shows standard "upper flange cleat" for 14″ × 12″ section.

Fig. B shows standard "upper flange cleat" for 10″ × 10″ section.

Fig. C shows suitable cleat for joining a 14″ × 12″ beam to a 10″ × 10″ stanchion.

FIG. C.



CONNECTIONS OF GIRDERS AT RIGHT ANGLES.

The following sketches show various ways of attaching two beams at right angles.

The angles to the lower flanges of the cross girders in Figs. E and F are contrivances for easy erection which add to the cost without greatly improving the construction. The "joggled" connection shown in Fig. J should be avoided as needlessly expensive.

FIG. D.

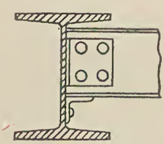


FIG. E.

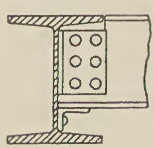


FIG. F.

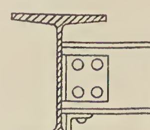


FIG. G.

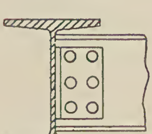


FIG. H.

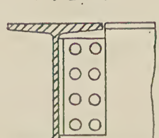
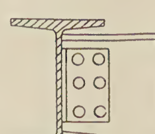


FIG. J.



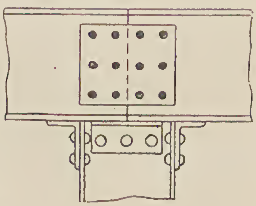
BROAD FLANGE BEAMS AS "BINDERS."

The smaller sections, when fitted with the standard web and flange cleats, make admirable "binders" or tie-beams, as they offer a greater average resistance in every direction than any other rolled sections obtainable.

Light joists of ordinary sections are less efficient, as they do not permit of satisfactory flange connections, and offer much less resistance to lateral flexure and compression.

FISHPLATES (Fig. 4).

The drawings show a pair of ordinary fishplates as used to join the ends of two girders resting on the same stanchion, as in sketch.



The "standard" fishplates have been made similar, in dimensions and in the number and arrangement of the bolts, to the standard "web cleats" (Figs. 1 and 2).

Fishplates are intended in cases of this kind to contribute to the general stability of the structure, and consequently there are no precise rules governing their design.* Incidentally, they stiffen the web and offer a measure of resistance to a possible tendency on the part of either beam to twist out of line.

Owing to difficulties of transport, very long girders have sometimes to be shipped in two pieces and spliced on the work, the joint being as far away from the centre of the span as possible.

In such cases, suitable cover plates are riveted to the flanges as well as to the web.

STANCHION BASES (Figs. 5, 7, and 8).

(1) The standard bases are all of sufficient area to enable them to be placed on suitable stone foundations. The nature of the foundation must depend, of course, on the load carried by the stanchion. First, divide the estimated load on the stanchion (tons) by the area (square feet) of the steel baseplate, as given in the foot-notes to the drawings. This will, of course, give the pressure in tons per square foot on the surface of the foundation, which can then be designed of suitable materials and dimensions from the data given on pages 176 and 177 ("Foundations").

(2) It is hardly economical to make the bases of a greater area than they are shown in this book; so that, if it is desired to avoid the use of stone templates, grillage foundations (see page 178) or cast-iron bases should be used.

(3) As regards thickness of metal, number and arrangement of rivets etc., the bases have been proportioned to the safe loads given in Table A† for

---

* A suitable thickness for fishplates can be ascertained by the following formula:—

Let H = effective depth of web, T = web thickness, B = width of fishplates (B will naturally be less than H), then required thickness of each plate = $T\ H^3 \div 2\ B^3$.

Or, more simply, each plate may be made of a thickness equal to about ¾ths of the thickness of the web.

The requisite number of bolts may be determined by providing a total bolt area on each side of the joint equal to one-half of the sectional area of the web. Or, to take a more scientific basis, a sufficient number of bolts may be provided to make their total bearing value on the web (or value in double shear, whichever is the less) approximately equal to the strength of the nett section of the web in shear.

† This refers to the General Table of Safe Loads on page 156.

stanchions 10 feet high. They may be regarded as equally suitable for stanchions of a greater height, as although long stanchions have smaller loads to carry, the value of fixing the ends as rigidly as practicable becomes increasingly important as the height of the stanchion is increased.

(4) The standard "bases" have been designed with due regard to the fact that the heavier stanchions may rest on three grillage joists as in Fig. 1, page 178, instead of having the pressure distributed evenly over the whole area of the baseplate, as when resting on a suitable stone foundation.

(5) If the load transmitted by the stanchion base does not exceed 25 to 30 tons per square foot, the base might rest directly on a reinforced concrete foundation, if designed and executed by experts in ferro-concrete.

(6) If the foundation is relatively deep, the base of the stanchion may be tied to the foundation by holding-down bolts embedded in the concrete. Particulars of suitable bolts will be found in the "List of Materials" (fourth page). Some engineers practically omit this precaution, considering that the load on the stanchion sufficiently ensures its stability.

(7) Cast-iron bases for stanchions composed of Broad Flange Beams would be about 1 foot deep, and the outside dimensions would be about $1\frac{3}{4}$ times the length and width of the standard steel bases.

N.B.—The remarkable way in which wide-flanged sections lend themselves to excellence and simplicity of connections is very noticeable in designing stanchion bases for Broad Flange Beams.

Built-stanchions do not provide sufficient flange area for double rows of rivets, and very often $\frac{3}{4}''$ rivets have to be used in cases where Broad Flange Beams of similar weight would permit the use of $\frac{7}{8}''$ or 1″ rivets.

In consequence of this fact, it is necessary when designing bases for built-stanchions, to extend the gusset angles and plates to a much greater height up the shaft in order to obtain end-fixing of anything like the same value as that furnished by the bases illustrated in this book.

For the same reason, gusseting has to be resorted to in practically all cases, whereas only the largest sizes of Broad Flange Beams require anything more than two pairs of angles and a soleplate.

As a consequence, the standard bases for Broad Flange Beams weigh less and cost less than similar bases to built-stanchions, although the latter are usually designed of considerably less carrying value than those illustrated in this book. A further advantage of wide-flanged stanchions, in this respect, is that the bases can be buried without carrying the foundations to an unreasonable depth, thus greatly adding to the value of the end-fixing without encroaching on the basement area.

STANCHION CAPS (Figs. 5 to 7).

Two patterns of "cap" are shown for each stanchion—a "light" pattern (Figs. 5 and 7) and an alternative "heavy" pattern (Fig. 6). The latter provides stronger flange cleats, but otherwise resembles the "light" pattern, except that the "heavy" caps for the smaller sections have been made 12″ wide, so that they will be suitable for any section of girder with 12″ flanges (*i.e.* any Broad Flange Beam from 12″ × 12″ upwards).

If the construction is of the character illustrated in Fig. A opposite, the following considerations will decide whether either of the standard "caps" is

suitable, and if so, which. The same procedure may be adopted if the construction is of the type shown in Fig. C.*

(1) First ascertain the end-reaction; that is, the load transmitted by the girder. If the girder carries a distributed load of 50 tons, the end-reaction will, of course, be 25 tons.

[The end-reactions corresponding to some other simple cases of loading can be ascertained by the formulæ on page 124.]

(2) Then each flange cleat in the stanchion cap should have a carrying power equal to not less than one-half of the end-reaction.

[The carrying power of these flange cleats is stated in the notes at the foot of the various drawings; it is represented by the value of the rivets in single shear, taken at 4 tons per square inch.]

(3) The remainder of the load may be assumed to be carried on the bolts by which the girder is fastened, through the web cleats, to the foot of the upper stanchion, and partly also on the cover plate if used. The shearing value of these bolts, taken at 4 tons per square inch, will be found on referring to the drawings relating to the *girder*. [That is: refer to Fig. 1 of the drawings relating to the girder section, and take one-half of the "distributed load" to which the web cleats are stated to be "suited."] As a rule, it will be found that these bolts would safely sustain the whole or nearly the whole of the load, and this is as it should be. In other words, it is desirable to assume that not more than one-half of the bolts come in shear.†

FIG. A.

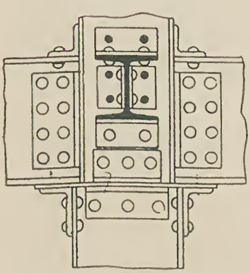


FIG. B.

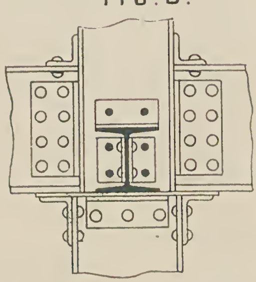
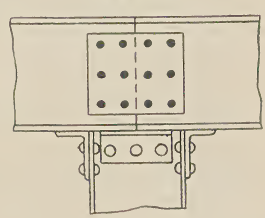


FIG. C.



* In Fig. C the girders have a direct bearing on the stanchion itself, the chief function of the flange cleats being to fix the upper end of the stanchion. Care should be paid to the fastenings, as the stanchions have obviously no lateral support other than that furnished by the resistance of the girders to twist or side deflection. This resistance is, of course, considerable in the case of wide-flanged girders, but the conditions approximate, in any case, to those of a strut fixed at one end and free at the other. On this account, if the load on the stanchion is comparatively small, it may be necessary to employ a stanchion nominally equal to two or three times the actual load in order to provide adequate fastenings, and to ensure the stability of the arrangement as a whole.

† The fastenings to the web and upper flanges of the girders in Fig. A contribute to the end-fixing of the upper stanchion, stiffen the girders, and also tend to eliminate tensile stresses in the rivets through the lower flange cleats. That is to say, if the girders were merely supported on the lower flange cleats, the rivets would be in combined shear and tension, owing to the eccentricity of the load. This is a point of some importance owing to the contraction of rivets in cooling and consequent tension. For this reason, fastenings should be carefully designed with a view to avoiding any further tensile stresses in rivets.

DRAWINGS OF STANDARD CONNECTIONS.—Continued.

COVER PLATE IN STANCHION CAP.

Cap plates are not usually required except in cases where one stanchion carries another stanchion of a smaller section above, as in Fig. A (page 17). [In this case the cap plate also relieves the fastenings to the lower stanchion of a part of the load transmitted by the girders, its carrying power being approximately equal to its strength in shear.] If the two stanchions are of the same section, or if the stanchion is terminated under girders, as in Fig. C, cap plates are not necessary.

WEB CLEATS IN STANCHION CAP.

The angles riveted to the web are similar to the web cleats shown for a girder end-fastening. Their principal function is to support the cap plate or they may be used to connect the webs of two stanchions as in Fig. A on page 17.

BINDERS TO STANCHIONS.

In the type of construction illustrated on page 22 (typical joint), the floor load is carried on the main girders fastened to the flanges of the stanchions, and the latter are braced laterally by " binders " or tie-beams.*

The lighter sections of Broad Flange Beams make excellent " binders " (see page 15), but in many cases still lighter H sections will be quite sufficient.

These " binders," in building construction, should be of an H section of which the least radius of gyration (inches) is equal to one-twentieth to one-tenth of its length in feet, according to the nature of the construction.

If the " binders " also carry a part of the load, they will naturally be proportioned as ordinary girders ; but, if they are relatively light compared with the principal girders, regard should none the less be paid to their value as compression-members, giving preference to as wide-flanged a section as the stanchions provide room for.

These tie-beams can rest on the cap plates as shown in Fig. B (page 17), or, if it is desired to make them level at the top with the main girders, the construction can be modified as in Fig. A.

BOLT-HOLES IN STANCHION CAP.

Holes for connection bolts are not shown in the caps as they must obviously be varied according to the size of the girders supported thereon. For example, if an 18″ × 12″ girder is supported on a 12″ × 12″ stanchion, the bolt-holes through the cap of the latter will naturally be arranged in the same manner as in the " lower flange cleat " shown in Figs. 1 and 2 of the drawings devoted to 18″ × 12″ section.

SECTIONS 12″ × 12″ AND UPWARDS.

It is a convenient feature of Broad Flange Beams that all sections from 12″ × 12″ upwards have a uniform flange width. Consequently, if the stanchion is of 12″ section or larger and carries girders of a heavier section, the flanges will be flush as in the illustration on page 23.

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* In bridge-work etc., compression-members for wind-bracing would usually have a least radius of gyration (inches) equal to about one-tenth of the length (feet).

LISTS OF MATERIALS.

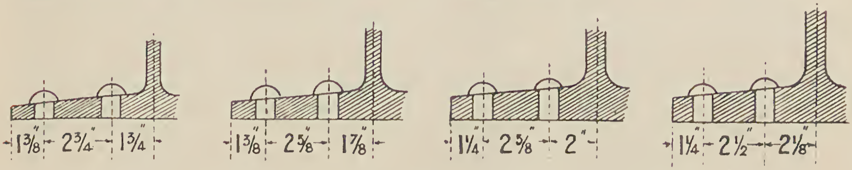
SIZES OF ANGLES USED.

The available range of sizes of angles is considerable. But a great many of these are used principally in shipbuilding and cannot readily be obtained in the small quantities required in building construction. For this reason, as few sizes of angles have been used as possible and only such as are frequently rolled by nearly all British manufacturers. The sizes used are as follows:—

$3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	$7'' \times 4'' \times \frac{5}{8}''$ and $\frac{3}{4}''$
$4'' \times 4'' \times \frac{1}{2}''$ and $\frac{5}{8}''$	$8'' \times 4'' \times \frac{5}{8}''$
$6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	$9'' \times 4'' \times \frac{1}{2}''$, $\frac{5}{8}''$ and $\frac{3}{4}''$
$6'' \times 4'' \times \frac{1}{2}''$, $\frac{5}{8}''$ and $\frac{3}{4}''$	

STANDARD PITCH OF RIVETS AND BOLTS.

Sections 12″ to 13½″ Sections 14″ to 16″ Sections 17″ to 20″ Sections 22″ to 30″



The above sketches show the pitch of bolts and rivets ($\frac{7}{8}''$ diameter) adopted for sections $12'' \times 12''$ and upwards.

The reasons for varying the pitch are that the rivet heads should be not less than $\frac{1}{4}''$ clear of the curve in any case; while, on the other hand, it is preferable not to space rivets closer than about 3 diameters centre to centre in the relatively thin flanges of the smaller sections.

VALUES OF RIVETS AND BOLTS.

For tables of Safe Shearing and Bearing Values of Rivets and Bolts, see page 214, Part II. ("Miscellaneous Tables.")

LISTS OF MATERIALS.

The particulars given on the fourth page, opposite to the drawings of standard connections, will facilitate drawing up specifications and quantities for estimating purposes.

The most useful form of specification for the latter purpose is one which gives a list of the various girders and stanchions in their finished condition prior to erection, after the following style:—

Item Nos.		Description.	Weight.		Price per ton.
1-14	14	Stanchions in basement, each consisting of etc. etc. - - -			
15-42	28	Girders in ground floor, each composed of etc. etc. - -			

The "Description" of the various items may be more or less detailed according to circumstances, but the following data should always be given, separately, for each item:—

(1) The SIZE (*i.e.* height, width, weight per foot, and/or section number) and the LENGTH of the rolled steel beam of which the stanchion or girder is composed.

(2) Whether any angles, plates etc. have to be riveted to the beam.
(3) Whether the beam has to be notched or drilled for bolts etc.

The "Item Nos." are arrived at by assigning consecutive numbers to all the stanchions and girders in the building, starting from the bottom. In the above example, the 14 stanchions are supposed to be exactly similar and are accordingly grouped together as a single entry in the schedule.

The "Weights" will naturally be given in the same degree of detail as the "Description." In any case, the following weights should be stated for each entry in the schedule:—

(1) Nett calculated weight of plain rolled steel beam of which member is composed.
(2) Gross weight of finished member, including angles, plates etc., but not bolts.

A list of bolts, fishplates, and other loose parts should usually be given separately at the end of the schedule, as otherwise such items are apt to be entered twice over.

A specification of steelwork in the above form can be priced more or less accurately without reference to the drawings. Sometimes the whole of the workmanship is stated at the end of the specification, but the above is a better plan, as the cost of workmanship varies greatly according to the size and weight of the pieces to be handled.

The following suggested stipulations may be found useful in drawing up general specifications.

BROAD FLANGE BEAMS.

(1) The rolled steel beams described as Broad Flange Beams must be produced in a mill of the type known as the "Grey Mill." Similar sections rolled in an ordinary mill with horizontal rolls will not be accepted. [N.B.—See page 222, "Grey Mill."]

(2) The dimensions of Broad Flange Beams are to be in strict accordance with the standard dimensions as published in H. J. Skelton & Co.'s Handbook (1906), subject to the customary rolling margins.

[Connections or fittings to Broad Flange Beams described as "Standard" are to be in accordance with the drawings of "Standard Connections" published in the aforesaid handbook.]

TESTS Etc.

All sectional material to be of mild steel of good uniform quality having an ultimate tensile strength of not less than 26 tons per square inch, and showing an elongation of not less than 20% in 8 inches. Rolled steel beams must be well and cleanly rolled, straight, true to section and free from twist throughout.*

[N.B.—In the case of relatively small orders, intended to be delivered from stock, the words "not less than 26 tons" should be replaced by the words "about 26 tons." *See notes on "Tests," page* 230, *Part II.*]

MACHINED ENDS.

All ends of beams used as stanchions are to be machined true and square in a fraizing or ending machine. All abutting surfaces are to be finished smooth and square and, if necessary, machined for this purpose after the end-fittings have been riveted on.

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* The specification should state whether the buyer intends appointing a representative or inspecting engineer to conduct tests at maker's works, or whether the tests are to be carried out by the manufacturer, and a certificate supplied showing the results thereof.

LISTS OF MATERIALS—Continued.

BOLTS AND NUTS.

Where not otherwise stated, these are to be of Whitworth standard pattern with hexagon heads and nuts. The stated lengths of ordinary bolts are measured from under head to point. Bolt-heads and nuts which bed on the inner flanges of rolled steel beams and other tapered surfaces are to be provided with suitable taper washers. [Dales's patent universal bolts and nuts make a better connection in such cases and are not more expensive. At present, however, they are not stocked, and should only be specified in lots of about 10 cwts. or more of each diameter and length.]

RIVETS.

Where not otherwise stated, rivets are to be cup-headed, the heads to be formed from a length equal to not less than $1\frac{1}{2}$ diameters. But rivets are to be flush countersunk where necessary. All rivets are to be machine-closed as far as practicable.

BOLT AND RIVET HOLES.

Bolts are to be of the full diameter indicated on drawings. Bolt-holes are to be of a diameter $\frac{1}{16}$th of an inch larger than the diameter of the bolts; except that, in the case of stanchion bases shipped separately from the shafts and bolted-up on arrival, the bolts must be turned to a driving fit, before shipment.

Holes for bolts must be set out and prepared by methods of sufficient accuracy to render reaming or drifting during erection unnecessary. Rivet holes may be $\frac{1}{16}$th of an inch larger than the rivets, but the latter must completely fill the holes when closed.

Holes in steel beams to be either (1) drilled true to size, or (2) punched small and afterwards reamed out not less than $\frac{1}{8}$th of an inch all round to the required diameter.

PAINTING.

The whole of the finished work to be cleaned from scale or rust and painted with one coat of good red oxide before despatch from works. [N.B.—If intended for shipment abroad, it is desirable to specify that the whole of the work shall receive one coat of boiled linseed oil, applied hot, before painting. If compound girders are employed, it is desirable to specify that all inaccessible parts be painted two coats before assemblage. Bolts, nuts, and rivets are sometimes ordered to be dipped into hot, boiled linseed oil.]

MISCELLANEOUS.

The following general clauses are sometimes useful:—

(1) The drawings and specification are to be read in conjunction; the accidental omission from the specification of work or materials clearly required by the drawings, or *vice versa*, shall not entitle the contractor to extra payment for such.

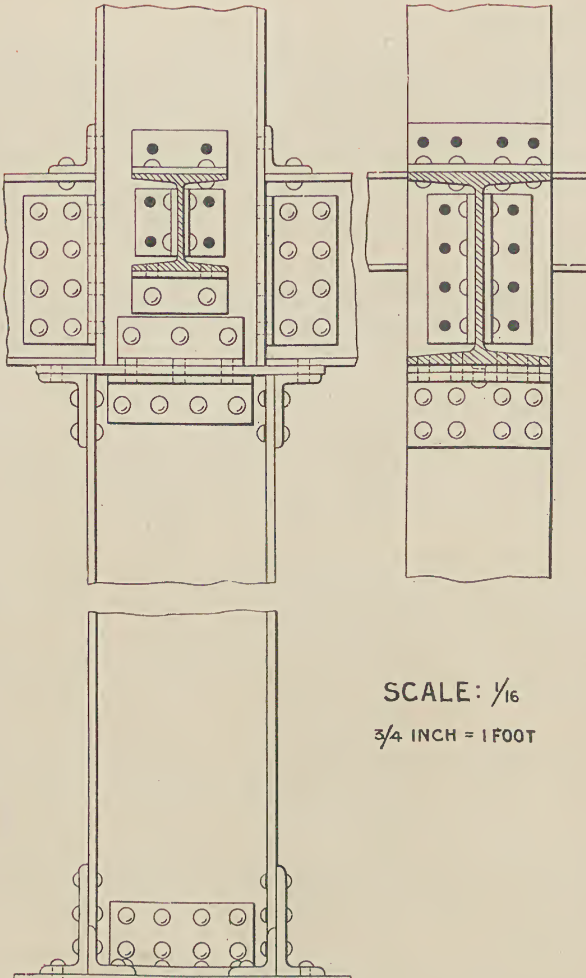
(2) The whole of the work is to be shipped in condition ready for immediate erection on arrival; all accessory parts to be riveted on as far as practicable before shipment.

(3) All requisite bolts and nuts to be supplied suitably packed and labelled, allowing not less than 10 % additional bolts and nuts of each size and length over and above actual number required. [The same remark will apply to loose rivets. It is necessary to state whether bolts or rivets are to be supplied for assemblage at site.]

(4) All members are to be suitably marked for convenience in erection in accordance with a key plan to be furnished at the time of or before shipment.

BROAD FLANGE BEAMS.

TYPICAL JOINT.



SCALE: 1/16

3/4 INCH = 1 FOOT

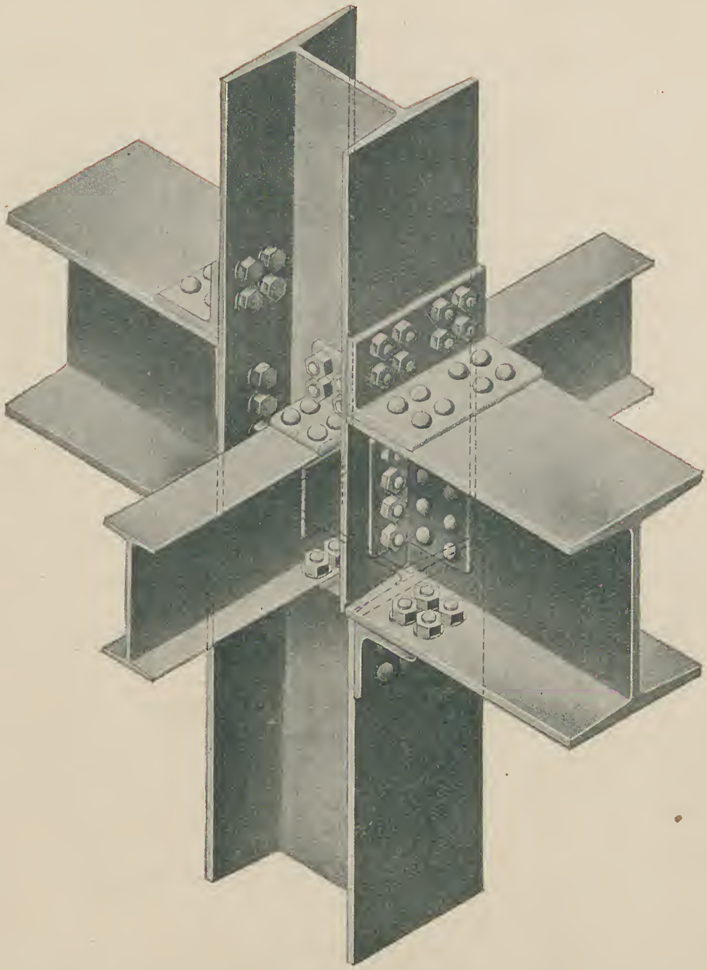
The above drawings show a typical joint in B.F. Beam construction (in side and end elevation).

The steelwork for warehouses of six or seven stories can be constructed in this fashion, using only plain rolled steel beams throughout. The lower stanchions in taller buildings can be made of B.F. Beams compounded or strengthened by plates riveted to the flanges, still preserving the essential features and advantages of the construction.

It will be observed that the lower end of each stanchion is stepped into the floor to the full depth of the flooring and main girders, the ends of the stanchions being secured to their utmost value in head and foot, thus making the height of a building a matter of little moment.

BROAD FLANGE BEAMS.

TYPICAL JOINT.



The above isometrical drawing represents a joint similar to that shown on the opposite page.

As regards details of construction, the drawings on the opposite page should be followed, but the above drawing shows more clearly the remarkable strength and simplicity of fastenings that B.F. Beams permit of. The characteristic features of structural work composed of wide-flanged sections are:—

(1) The individual girders and stanchions are lighter and cheaper; time, weight and cost being saved throughout.

(2) The possibility of using double rows of rivets where, in riveted girders and stanchions, only a single row of smaller rivets could be used, renders the connections extremely simple and economical.

(3) Besides providing much more simply for the vertical loads, the fastenings further contribute lateral stability to the stanchions and a degree of rigidity to the general structure far exceeding that obtainable from the best possible fastenings to all other types of girders and stanchions. (Compare the fastenings to the flanges of the girders in the above drawing with similar fastenings to ordinary joists and riveted sections, noting also the lateral stiffness of the girders themselves.)

(4) Accessibility of every part of the structure for painting etc.

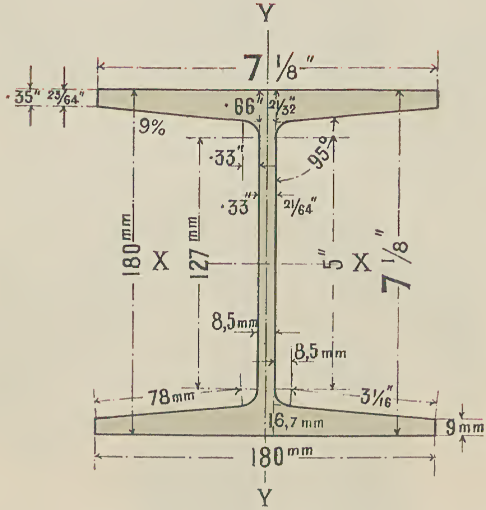
(5) Convenience in erection, economising time and labour. Every nut and bolt-head is freely accessible to ordinary tools and the wide-flanged members are convenient to handle.

(6) Diminished liability to rust as compared with plate and angle girders and other built-up members.

SECTION DRAWING OF

B.F. BEAM { Section No. 104. Code Word: "ABBESS."
Nominal Dimensions 7″ × 7″ × 31½ lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM { Section No. 104. Code Word: "ABBESS."
Nominal Dimensions 7″ × 7″ × 31½ lbs. per foot.
Actual Dimensions 7⅛″ × 7⅛″ × 31½ lbs. per foot. }

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=71 feet to the ton, approx.)	31½ lbs. per foot	47·0 kilos. per m.
Sectional Area	9·3 sq. ins.	59·9 sq. cm.
Greatest Moment of Inertia (Axis XX)	84·4 inches^4	3512 cm^4
Least ,, ,, (Axis YY)	25·8 ,,	1073 ,,
Greatest Section Modulus (Axis XX)	23·8 inches^3	390 cm^3
Least ,, ,, (Axis YY)	7·27 ,,	119 ,,
Moment of Resistance (xx) at 7½ tons stress	179 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	23 tons (5·1 ft.)	
Greatest Radius of Gyration (Axis XX)	3·01 inches	
Least ,, ,, (Axis YY)	1·67 ,,	
Maximum length rolled	about 68 feet	about 21 metres

The "MOMENT OF RESISTANCE" is the "Greatest Section Modulus" multiplied by the working stress (here taken as 7½ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: 7½ tons per square inch).

Span (feet)	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons)	15	12	10	8	7	6	6	...	...	...	...	...	...	...
Deflection (inches)	·16	·25	·36	·49	·64	·80	·99	...	...	...	...	...	...	...

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about 9 inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·17 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet)	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons)	43	42	40	38	36	34	31	29	27	25	24	22	20	18
Nett Weight (cwts.)	2·26	2·54	2·82	3·10	3·38	3·67	3·95	4·23	4·51	4·79	5·08	5·36	5·64	6·20

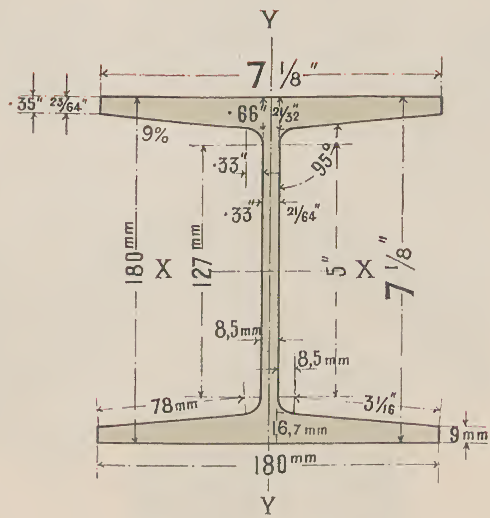
Weight of Stanchion Base, about 1·28 cwts.; Cap, about 0·49 cwt.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·38 tons if applied to one of the flanges, or 1·21 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

SECTION DRAWING OF

B.F. BEAM { Section No. 104. Code Word: "ABBESS."
Nominal Dimensions 7″ × 7″ × 31½ lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM Section No. 104. Code Word: "ABBESS."
Nominal Dimensions 7" × 7" × $31\frac{1}{2}$ lbs. per foot.
Actual Dimensions $7\frac{1}{8}$" × $7\frac{1}{8}$" × $31\frac{1}{2}$ lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=71 feet to the ton, approx.)	$31\frac{1}{2}$ lbs. per foot	47·0 kilos. per m.
Sectional Area	9·3 sq. ins.	59·9 sq. cm.
Greatest Moment of Inertia (Axis XX)	84·4 inches^4	3512 cm^4
Least " " (Axis YY)	25·8 "	1073 "
Greatest Section Modulus (Axis XX)	23·8 inches^3	390 cm^3
Least " " (Axis YY)	7·27 "	119 "
Moment of Resistance (xx) at $7\frac{1}{2}$ tons stress	179 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	23 tons (5·1 ft.)	
Greatest Radius of Gyration (Axis XX)	3·01 inches	
Least " " (Axis YY)	1·67 "	
Maximum length rolled	about 68 feet	about 21 metres

The "Moment of Resistance" is the "Greatest Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: $7\frac{1}{2}$ tons per square inch).

Span (feet)	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons)	15	12	10	8	7	6	6	...	...	...	...	...	...	...
Deflection (inches)	·16	·25	·36	·49	·64	·80	·99	...	...	...	...	...	...	...

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about 9 inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·17 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet)	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons)	43	42	40	38	36	34	31	29	27	25	24	22	20	18
Nett Weight (cwts.)	2·26	2·54	2·82	3·10	3·38	3·67	3·95	4·23	4·51	4·79	5·08	5·36	5·64	6·20

Weight of Stanchion Base, about 1·28 cwts.; Cap, about 0·49 cwt.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·38 tons if applied to one of the flanges, or 1·21 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 104. Code Word: "ABBESS."
Nominal Dimensions $7'' \times 7'' \times 31\frac{1}{2}$ lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

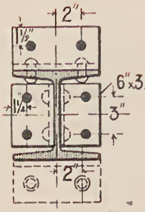


Fig. 1

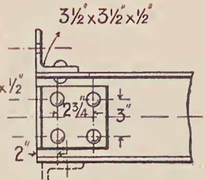


Fig. 2

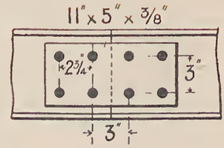


Fig. 4

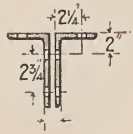


Fig. 3

SCALE 1/16

($\frac{3}{4}$ inch = 1 foot)

Rivets and Bolts $\frac{3}{4}''$

Bolt-Holes $\frac{13}{16}''$

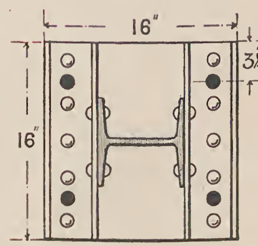


Fig. 8

Fig. 5

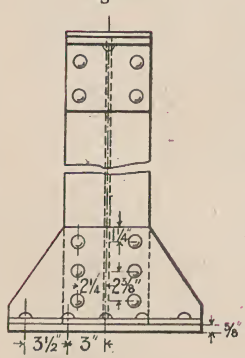


Fig. 6

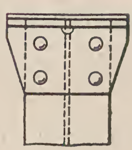
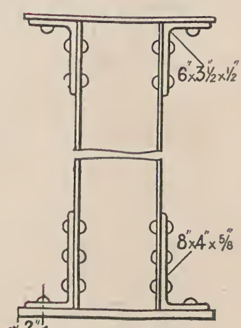


Fig. 7



Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 17·7 tons (span: 6·7 feet). The "Web Cleats" by themselves are suited to a distributed load of 14·1 tons only (span: 8·4 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, 1·78 square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 7·1 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 7·1 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 104. Code Word: "ABBESS."
Nominal Dimensions $7'' \times 7'' \times 31\frac{1}{2}$ lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.	LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 24.	Broad Flange Beam(s) $7'' \times 7''$ approx. -	$31\frac{1}{2}$ lbs. per ft.
	H. J. SKELTON & Co.'s Section No. 104	
	N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	71 ft. per ton
Figs. 1-3. Web Cleats	2 Angles $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by 5″ long - -	13 lbs. per pair
	4 $\frac{3}{4}''$ Rivets, $1\frac{3}{8}''$ grip - - - - - -	2 lbs.
	4 $\frac{3}{4}''$ Bolts - - - - - - -	$3\frac{1}{2}$ lbs. say
	Gross weight of Web Cleats	$18\frac{1}{2}$ lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1 Angle $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by 7″ long - -	$6\frac{1}{2}$ lbs.
	2 $\frac{3}{4}''$ Rivets, 1″ grip - - - - - -	1 lb.
	2 $\frac{3}{4}''$ Bolts - - - - - - -	2 lbs. say
	Gross weight of one Upper Flange Cleat	$9\frac{1}{2}$ lbs.
Figs. 1, 2. Lower Flange Cleat	Same as "Upper Flange Cleat" - -	$9\frac{1}{2}$ lbs.
Fig. 4. Fishplates	2 $11'' \times 5'' \times \frac{3}{8}''$ - - - - - - -	12 lbs. per pair
	8 $\frac{3}{4}''$ Bolts, 2″ long - - - - - -	$5\frac{1}{2}$ lbs.
	Gross weight of Fishplates	$17\frac{1}{2}$ lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2 Angles to flanges $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by 7″ long	18 lbs. per pair
	1 Cover Plate (if required) $16'' \times 7'' \times \frac{3}{8}''$ -	12 lbs.
	10 $\frac{3}{4}''$ Rivets (2 countersunk) - - -	$4\frac{1}{2}$ lbs.
	4 $\frac{3}{4}''$ Bolts and Nuts - - - - -	4 lbs. say
	Gross weight of one Light Cap (including plate)	$38\frac{1}{2}$ lbs.
Fig. 6. Stanchion Cap (heavy)	2 Angles to flanges $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by 10″ long	$25\frac{1}{2}$ lbs. per pair
	1 Cover Plate (if required) $16'' \times 10'' \times \frac{3}{8}''$ -	17 lbs.
	10 $\frac{3}{4}''$ Rivets (2 countersunk) - - -	$4\frac{1}{2}$ lbs.
	(8) $\frac{3}{4}''$ Bolts and Nuts - - - - - -	8 lbs. say
	Gross weight of one Heavy Cap (including plate)	55 lbs.
Figs. 5-8. Stanchion Base	1 Sole Plate $16'' \times 16'' \times \frac{5}{8}''$ - - - - -	45 lbs.
	2 Angles to flanges $8'' \times 4'' \times \frac{5}{8}''$ by 16″ long	$64\frac{1}{2}$ lbs. per pair
	22 $\frac{3}{4}''$ Rivets (10 countersunk) - - -	$9\frac{1}{2}$ lbs.
	4 Holding-down Bolts $1'' \times 18''$ with $3'' \times 3'' \times \frac{1}{4}''$ plates - - - -	24 lbs.
	Gross weight of one Stanchion Base .	143 lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 4 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

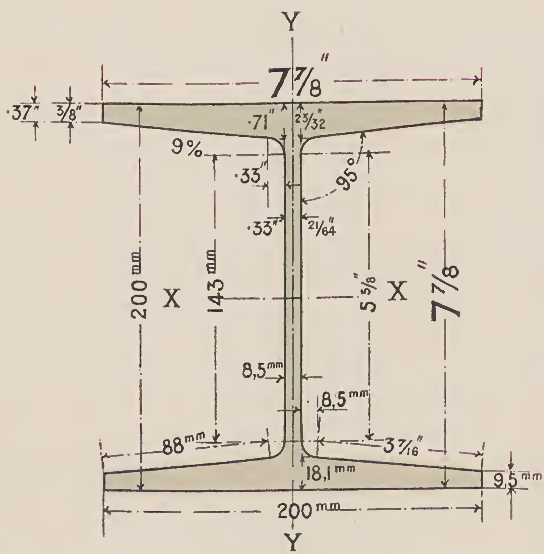
For further notes, see page 19.

SECTION DRAWING OF

B.F. BEAM { Section No. 108. Code Word : " ABODE."
Nominal Dimensions 8" × 8" × 37 lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]

STOCK SIZE.



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM
- Section No. 108. Code Word: "ABODE."
- Nominal Dimensions 8″ × 8″ × 37 lbs. per foot.
- Actual Dimensions $7\frac{7}{8}'' \times 7\frac{7}{8}''$ × 37 lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=60 feet to the ton, approx.) - -	37 lbs. per foot	55·3 kilos. per m.
Sectional Area - - - - - -	10·9 sq. ins.	70·4 sq. cm.
Greatest Moment of Inertia (Axis XX) - -	124 inches^4	5171 cm^4
Least ,, ,, (Axis YY) - -	37·7 ,,	1568 ,,
Greatest Section Modulus (Axis XX) - -	31·6 inches^3	517 cm^3
Least ,, ,, (Axis YY) - -	9·58 ,,	157 ,,
Moment of Resistance (XX) at $7\frac{1}{2}$ tons stress -	237 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	25 tons (6·3 ft.)	
Greatest Radius of Gyration (Axis XX) - -	3·38 inches	
Least ,, ,, (Axis YY) - -	1·86 ,,	
Maximum length rolled - - - - -	about 68 feet	about 21 metres

The "Moment of Resistance" is the "Greatest Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: $7\frac{1}{2}$ tons per square inch).

Span (feet) - -	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons) -	19	15	13	11	10	9	8	...	...	...	...	...	...	...
Deflection (inches)	·14	·22	·32	·44	·57	·73	·90	...	...	...	...	...	...	...

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about 9 inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·19 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet) -	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons) -	52	51	49	47	45	43	41	38	36	34	32	30	28	24
Nett Weight (cwts.)	2·65	2·98	3·31	3·64	3·97	4·31	4·64	4·97	5·30	5·63	5·96	6·29	6·62	7·29

Weight of Stanchion Base, about 1·47 cwts.; Cap, about 0·61 cwt.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·36 tons if applied to one of the flanges, or 1·19 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 108. Code Word: "ABODE."
Nominal Dimensions 8″ × 8″ × 37 lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

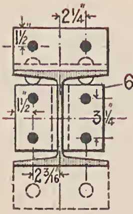


Fig. 1

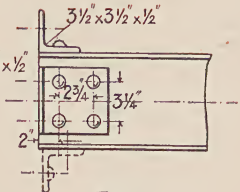


Fig. 2

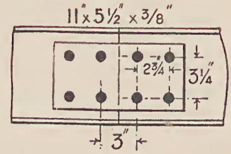


Fig. 4

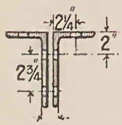


Fig. 3

SCALE 1/16

($\frac{3}{4}$ inch = 1 foot)

Rivets and Bolts $\frac{3}{4}$″

Bolt-Holes $\frac{13}{16}$″

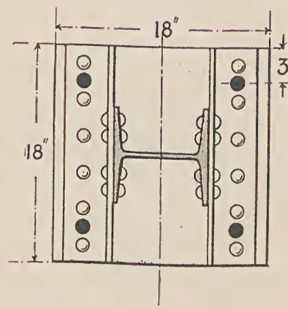


Fig. 8

Fig. 5

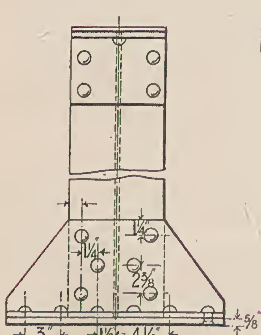


Fig. 6

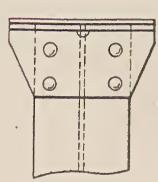
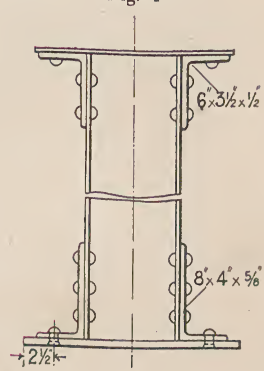


Fig. 7



Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 17·7 tons (span: 8·9 feet). The "Web Cleats" by themselves are suited to a distributed load of 14·1 tons only (span: 11·2 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, 2¼ square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 7·1 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 7·1 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 108. Code Word : "ABODE."
Nominal Dimensions $8'' \times 8'' \times 37$ lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.	LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 28.	Broad Flange Beam(s) $8'' \times 8''$ approx. - H. J. SKELTON & Co.'s Section No. 108	37 lbs. per ft.
	N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	60 ft. per ton
Figs. 1-3. Web Cleats	2 Angles $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by $5\frac{1}{2}''$ long - -	14 lbs. per pair
	4 $\frac{3}{4}''$ Rivets, $1\frac{3}{8}''$ grip - - - - -	2 lbs.
	4 $\frac{3}{4}''$ Bolts - - - - - - -	$3\frac{1}{2}$ lbs. say
	Gross weight of Web Cleats	$19\frac{1}{2}$ lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1 Angle $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by $8''$ long - -	$7\frac{1}{2}$ lbs.
	2 $\frac{3}{4}''$ Rivets, $1''$ grip - - - - -	1 lb.
	2 $\frac{3}{4}''$ Bolts - - - - - - -	2 lbs. say
	Gross weight of one Upper Flange Cleat	$10\frac{1}{2}$ lbs.
Figs. 1, 2. Lower Flange Cleat	Same as "Upper Flange Cleat" - -	$10\frac{1}{2}$ lbs.
Fig. 4. Fishplates	2 $11'' \times 5\frac{1}{2}'' \times \frac{3}{8}''$ - - - - - - -	13 lbs. per pair
	8 $\frac{3}{4}''$ Bolts, $2''$ long - - - - -	$5\frac{1}{2}$ lbs.
	Gross weight of Fishplates	$18\frac{1}{2}$ lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2 Angles to flanges $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by $8''$ long -	$20\frac{1}{2}$ lbs. per pair
	1 Cover Plate (if required) $16\frac{1}{2}'' \times 8'' \times \frac{3}{8}''$ -	14 lbs.
	10 $\frac{3}{4}''$ Rivets (2 countersunk) - - -	$4\frac{1}{2}$ lbs.
	4 $\frac{3}{4}''$ Bolts - - - - - - -	4 lbs. say
	Gross weight of one Light Cap (including plate)	43 lbs.
Fig. 6. Stanchion Cap (heavy)	2 Angles to flanges $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by $12''$ long	31 lbs. per pair
	1 Cover Plate (if required) $16\frac{1}{2}'' \times 12'' \times \frac{3}{8}''$ -	21 lbs.
	10 $\frac{3}{4}''$ Rivets (2 countersunk) - - -	$4\frac{1}{2}$ lbs.
	(8) $\frac{7}{8}''$ Bolts and Nuts - - - - -	12 lbs. say
	Gross weight of one Heavy Cap (including plate)	$68\frac{1}{2}$ lbs.
Figs. 5-8. Stanchion Base	1 Sole Plate $18'' \times 18'' \times \frac{5}{8}''$ - - - -	58 lbs.
	2 Angles to flanges $8'' \times 4'' \times \frac{5}{8}''$ by $18''$ long-	$72\frac{1}{2}$ lbs. per pair
	24 $\frac{3}{4}''$ Rivets (12 countersunk) - - -	10 lbs.
	4 Holding-down Bolts $1'' \times 18''$ with $3'' \times 3'' \times \frac{1}{4}''$ plates - - - -	24 lbs.
	Gross weight of one Stanchion Base	$164\frac{1}{2}$ lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 4 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

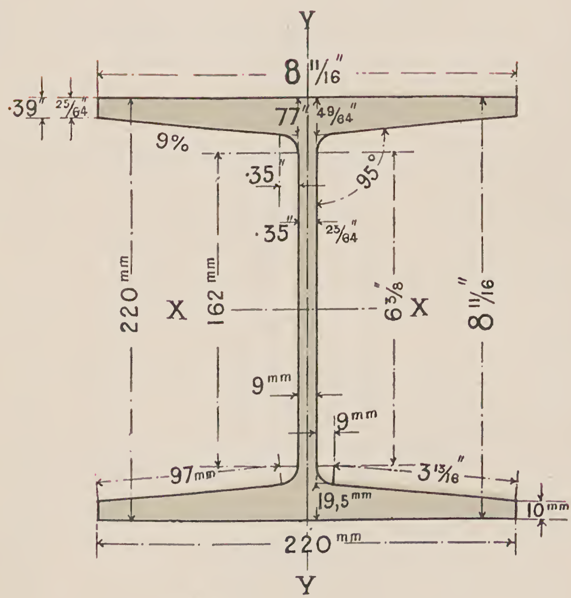
For further notes, see page 19.

SECTION DRAWING OF

B.F. BEAM { Section No. 112. Code Word: "ABSTAINER."
Nominal Dimensions $8\frac{1}{2}'' \times 8\frac{1}{2}'' \times 44$ lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]

STOCK SIZE.



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM { Section No. 112. Code Word: "ABSTAINER."
Nominal Dimensions $8\frac{1}{2}'' \times 8\frac{1}{2}'' \times 44$ lbs. per foot.
Actual Dimensions $8\frac{11}{16}'' \times 8\frac{11}{16}'' \times 44$ lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=51 feet to the ton, approx.) - -	44 lbs. per foot	64·8 kilos. per m.
Sectional Area - - - - - -	12·8 sq. ins.	82·6 sq. cm.
Greatest Moment of Inertia (Axis XX) - -	177 inches^4	7379 cm^4
Least ,, ,, (Axis YY) - -	53·3 ,,	2216 ,,
Greatest Section Modulus (Axis XX) - -	41·0 inches^3	671 cm^3
Least ,, ,, (Axis YY) - -	12·3 ,,	201 ,,
Moment of Resistance (XX) at $7\frac{1}{2}$ tons stress -	307 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	28 tons (7·3 ft.)	
Greatest Radius of Gyration (Axis XX) - -	3·72 inches	
Least ,, ,, (Axis YY) - -	2·04 ,,	
Maximum length rolled - - - - -	about 68 feet	about 21 metres

The "MOMENT OF RESISTANCE" is the greatest "Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: $7\frac{1}{2}$ tons per square inch).

Span (feet) - -	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons) -	26	20	17	15	13	11	10	9	7	...	...	...	...	...
Deflection (inches)	·13	·20	·29	·40	·52	·66	·81	1·2	1·6	...	...	...	...	...

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about 9 inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·20 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet) -	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons) -	62	61	59	57	55	53	51	49	46	43	41	39	36	32
Nett Weight (cwts.)	3·11	3·50	3·89	4·27	4·66	5·05	5·44	5·83	6·22	6·61	6·99	7·38	7·77	8·55

Weight of Stanchion Base, about 1·56 cwts.; Cap, about 0·63 cwt.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·35 tons if applied to one of the flanges, or 1·18 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 112. Code Word: "ABSTAINER."
Nominal Dimensions $8\frac{1}{2}'' \times 8\frac{1}{2}'' \times 44$ lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

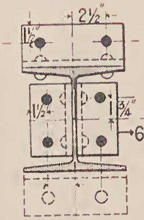


Fig. 1

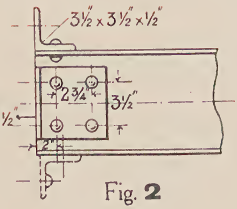


Fig. 2

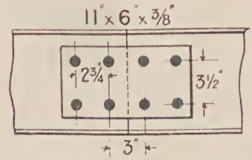


Fig 4

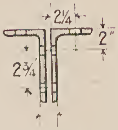


Fig. 3

SCALE 1/16

($\frac{3}{4}$ inch = 1 foot)

Rivets and Bolts $\frac{3}{4}''$

Bolt-Holes $\frac{13}{16}''$

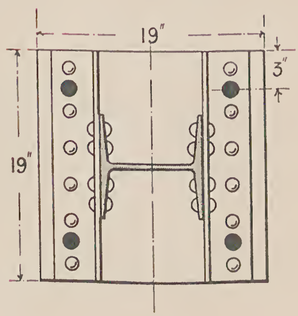


Fig. 8

Fig. 5

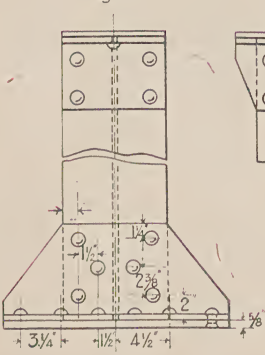


Fig. 6

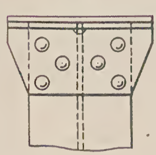
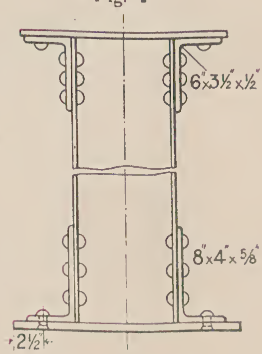


Fig. 7



Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 17·7 tons (span: 11·6 feet). The "Web Cleats" by themselves are suited to a distributed load of 14·1 tons only (span: 14½ feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, 2·51 square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 7·1 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 10·6 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 112. Code Word: "ABSTAINER."
Nominal Dimensions $8\frac{1}{2}'' \times 8\frac{1}{2}'' \times 44$ lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.	LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 32.	Broad Flange Beam(s) $8\frac{1}{2}'' \times 8\frac{1}{2}''$ approx. -	44 lbs. per ft.
	H. J. SKELTON & Co.'s Section No. 112	
	N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	51 ft. per ton
Figs. 1-3. Web Cleats	2 Angles $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by 6″ long - -	15 lbs. per pair
	4 $\frac{3}{4}''$ Rivets, $1\frac{3}{8}''$ grip - - - - -	2 lbs.
	4 $\frac{3}{4}''$ Bolts - - - - - - -	$3\frac{1}{2}$ lbs. say
	Gross weight of Web Cleats	$20\frac{1}{2}$ lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1 Angle $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by $8\frac{1}{2}''$ long - -	8 lbs.
	2 $\frac{3}{4}''$ Rivets, $1\frac{1}{8}''$ grip - - - - -	1 lb.
	2 $\frac{3}{4}''$ Bolts - - - - - - -	2 lbs. say
	Gross weight of one Upper Flange Cleat	11 lbs.
Figs. 1, 2. Lower Flange Cleat	Same as "Upper Flange Cleat" - -	11 lbs.
Fig. 4. Fishplates	2 $11'' \times 6'' \times \frac{3}{8}''$ - - - - - - -	14 lbs. per pair
	8 $\frac{3}{4}''$ Bolts, 2″ long - - - - -	$5\frac{1}{2}$ lbs.
	Gross weight of Fishplates	$19\frac{1}{2}$ lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2 Angles to flanges $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by $8\frac{1}{2}''$ long	22 lbs. per pair
	1 Cover Plate (if required) $17'' \times 8\frac{1}{2}'' \times \frac{3}{8}''$ -	15 lbs.
	10 $\frac{3}{4}''$ Rivets (2 countersunk) - - -	$4\frac{1}{2}$ lbs.
	4 $\frac{3}{4}''$ Bolts - - - - - - -	4 lbs. say
	Gross weight of one Light Cap (including plate)	$45\frac{1}{2}$ lbs.
Fig. 6. Stanchion Cap (heavy)	2 Angles to flanges $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by 12″ long	31 lbs. per pair
	1 Cover Plate (if required) $17'' \times 12'' \times \frac{3}{8}''$ -	22 lbs.
	14 $\frac{3}{4}''$ Rivets (2 countersunk) - - -	6 lbs.
	(8) $\frac{7}{8}''$ Bolts and Nuts - - - - -	12 lbs. say
	Gross weight of one Heavy Cap (including plate)	71 lbs.
Figs. 5-8. Stanchion Base	1 Sole Plate $19'' \times 19'' \times \frac{5}{8}''$ - - - -	64 lbs.
	2 Angles to flanges $8'' \times 4'' \times \frac{5}{8}''$ by 19″ long	$76\frac{1}{2}$ lbs. per pair
	24 $\frac{3}{4}''$ Rivets (12 countersunk) - - -	10 lbs.
	4 Holding-down Bolts $1'' \times 18''$ with $3'' \times 3'' \times \frac{1}{4}''$ plates - - - -	24 lbs.
	Gross weight of one Stanchion Base	$174\frac{1}{2}$ lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 4 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

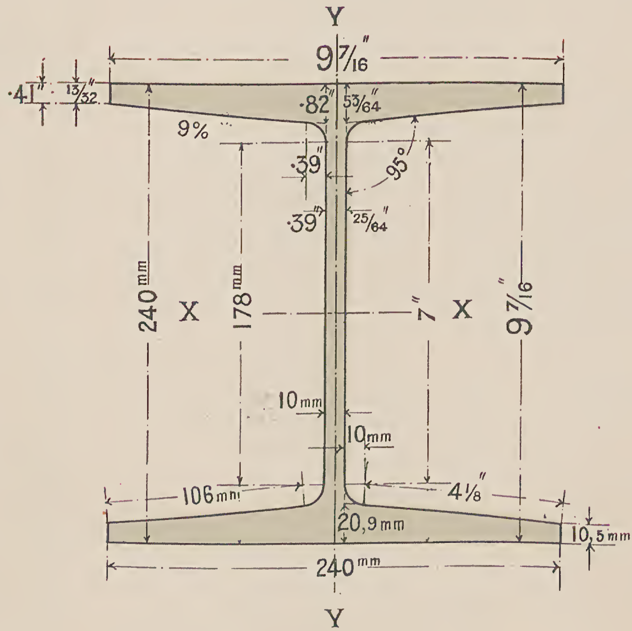
For further notes, see page 19.

SECTION DRAWING OF

B.F. BEAM { Section No. 116. Code Word: "ABYSS."
Nominal Dimensions $9\frac{1}{2}'' \times 9\frac{1}{2}'' \times 51$ lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]

STOCK SIZE.



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM — Section No. 116. Code Word: "ABYSS."
Nominal Dimensions $9\frac{1}{2}'' \times 9\frac{1}{2}'' \times 51$ lbs. per foot.
Actual Dimensions $9\frac{7}{16}'' \times 9\frac{7}{16}'' \times 51$ lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=44 feet to the ton, approx.) - -	51 lbs. per foot	76·0 kilos. per m.
Sectional Area - - - - - -	15·0 sq. ins.	96·8 sq. cm.
Greatest Moment of Inertia (Axis XX) - -	247 inches^4	10260 cm^4
Least " " (Axis YY) - -	73·1 "	3043 "
Greatest Section Modulus (Axis XX) - -	52·2 inches^3	855 cm^3
Least " " (Axis YY) - -	15·5 "	254 "
Moment of Resistance (XX) at $7\frac{1}{2}$ tons stress -	391 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	35 tons ($7\frac{1}{2}$ ft.)	
Greatest Radius of Gyration (Axis XX) - -	4·05 inches	
Least " " (Axis YY) - -	2·21 "	
Maximum length rolled - - - - -	about 68 feet	about 21 metres

The "Moment of Resistance" is the "Greatest Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: $7\frac{1}{2}$ tons per square inch).

Span (feet) - -	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons) -	33	26	22	19	16	14	13	11	9	8	...	...	...	...
Deflection (inches)	·12	·19	·27	·37	·48	·61	·75	1·1	1·5	1·9	...	...	...	...

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about $13\frac{1}{2}$ inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·30 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet) -	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons) -	74	72	71	69	67	65	63	61	58	55	52	49	47	42
Nett Weight (cwts.)	3·64	4·10	4·55	5·01	5·46	5·92	6·38	6·83	7·29	7·74	8·20	8·65	9·11	10·0

Weight of Stanchion Base, about 1·97 cwts.; Cap, about 0·65 cwt.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·36 tons if applied to one of the flanges, or 1·19 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

$9\frac{1}{2}''$ 10" $10\frac{1}{4}$ $10\frac{1}{2}''$ 11" $11\frac{1}{2}''$ 12" $12\frac{1}{2}''$ $13\frac{1}{2}''$ 14" 15" 16" 17 18" 19 20" 22" 24" 26" 30

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 116. Code Word: "ABYSS."
Nominal Dimensions $9\frac{1}{2}'' \times 9\frac{1}{2}'' \times 51$ lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

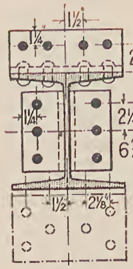


Fig. **1**

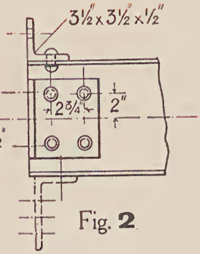


Fig. **2**

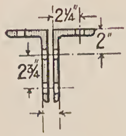


Fig. **3**

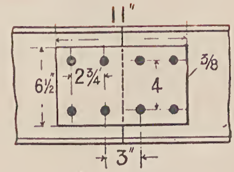


Fig. **4**

SCALE 1/16

($\frac{3}{4}$ inch = 1 foot)

Rivets and Bolts $\frac{3}{4}''$

Bolt-Holes $\frac{13}{16}''$

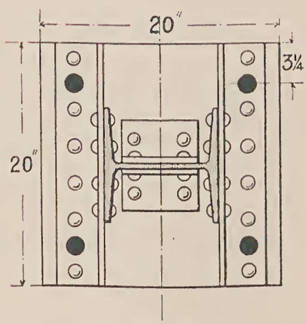


Fig. **8**

Fig. **5**

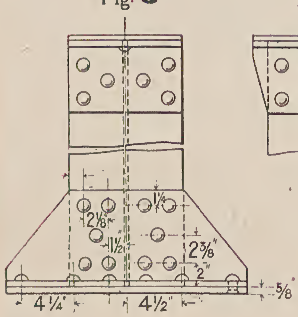


Fig. **6**

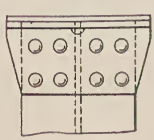
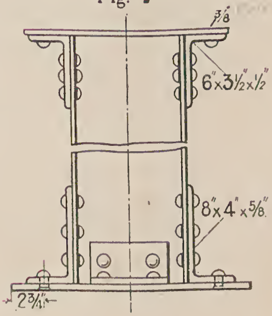


Fig. **7**



Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 37·1 tons (span: 7 feet). The "Web Cleats" by themselves are suited to a distributed load of 21·2 tons only (span: 12·3 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, 2·78 square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 10·6 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 14·1 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 116. Code Word: "ABYSS."
Nominal Dimensions $9\frac{1}{2}'' \times 9\frac{1}{2}'' \times 51$ lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.		LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 36.		Broad Flange Beam(s) $9\frac{1}{2}'' \times 9\frac{1}{2}''$ approx. - H. J. SKELTON & Co.'s Section No. 116	51 lbs. per ft.
		N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	44 ft. per ton
Figs. 1-3. Web Cleats	2	Angles $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by $6\frac{1}{2}''$ long - -	$16\frac{1}{2}$ lbs. per pair
	4	$\frac{3}{4}''$ Rivets, $1\frac{3}{8}''$ grip - - - - -	2 lbs.
	6	$\frac{3}{4}''$ Bolts - - - - - - -	$5\frac{1}{2}$ lbs. say
		Gross weight of Web Cleats	24 lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1	Angle $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by $9\frac{1}{2}''$ long - -	9 lbs.
	4	$\frac{3}{4}''$ Rivets, $1\frac{1}{4}''$ grip - - - - -	2 lbs.
	4	$\frac{3}{4}''$ Bolts - - - - - - -	4 lbs. say
		Gross weight of one Upper Flange Cleat	15 lbs.
Figs. 1, 2. Lower Flange Cleat	1	Angle $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by $9\frac{1}{2}''$ long -	12 lbs.
		Gross weight including 4 Bolts & 6 Rivets	19 lbs. say
Fig. 4. Fishplates	2	$11'' \times 6\frac{1}{2}'' \times \frac{3}{8}''$ - - - - - - -	15 lbs. per pair
	8	$\frac{3}{4}''$ Bolts, $2''$ long - - - - - -	$5\frac{1}{2}$ lbs.
		Gross weight of Fishplates	$20\frac{1}{2}$ lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2	Angles to flanges $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by $9\frac{1}{2}''$ long	24 lbs. per pair
	1	Cover Plate (if required) $17'' \times 9\frac{1}{2}'' \times \frac{3}{8}''$ -	17 lbs.
	14	$\frac{3}{4}''$ Rivets (2 countersunk) - - - -	6 lbs.
	(8)	$\frac{3}{4}''$ Bolts - - - - - - -	8 lbs. say
		Gross weight of one Light Cap (including plate)	55 lbs.
Fig. 6. Stanchion Cap (heavy)	2	Angles $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by $12''$ long - -	31 lbs. per pair
	1	Cover Plate (if required) $17'' \times 12'' \times \frac{3}{8}''$ -	22 lbs.
	18	$\frac{3}{4}''$ Rivets (2 countersunk) - - -	8 lbs.
	(8)	$\frac{7}{8}''$ Bolts and Nuts - - - - -	12 lbs. say
		Gross weight of one Heavy Cap (including plate)	73 lbs.
Figs. 5-8. Stanchion Base.	1	Sole Plate $20'' \times 20'' \times \frac{5}{8}''$ - - - - -	71 lbs.
	2	Angles to flanges $8'' \times 4'' \times \frac{5}{8}''$ by $20''$ long -	$80\frac{1}{2}$ lbs. per pair
	2	Angles to web $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by $6\frac{1}{2}''$ long -	12 lbs. per pair
	38	$\frac{3}{4}''$ Rivets (16 countersunk) - - - -	$16\frac{1}{2}$ lbs.
	4	Holding-down Bolts $1\frac{1}{4}'' \times 18''$ with $4'' \times 4'' \times \frac{3}{8}''$ plates - - - -	41 lbs.
		Gross weight of one Stanchion Base	221 lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 4 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

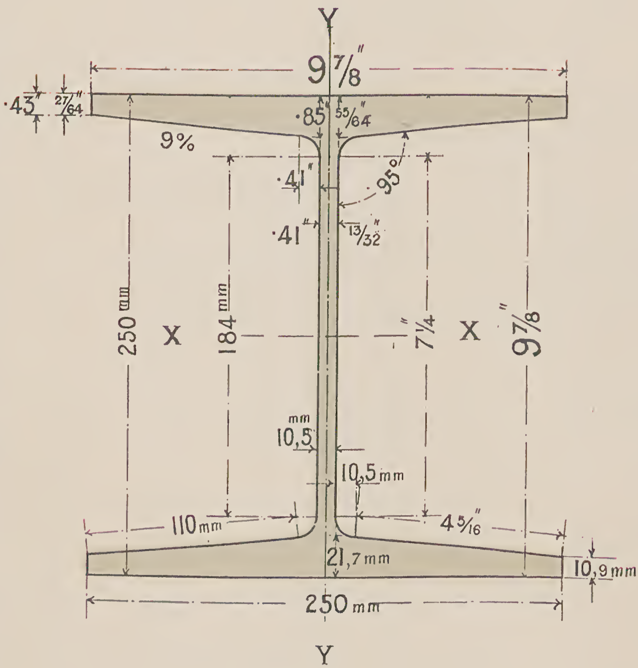
For further notes, see page 19.

SECTION DRAWING OF

B.F. BEAM { Section No. 120. Code Word: "ACCENT."
Nominal Dimensions 10″ × 10″ × 55 lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]

STOCK SIZE.



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM — Section No. 120. Code Word: "ACCENT."
Nominal Dimensions 10″ × 10″ × 55 lbs. per foot.
Actual Dimensions $9\frac{7}{8}''$ × $9\frac{7}{8}''$ × 55 lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=40 feet to the ton, approx.)	55 lbs. per foot	82·5 kilos. per m.
Sectional Area	16·3 sq. ins.	105·1 sq. cm.
Greatest Moment of Inertia (Axis XX)	290 inches^4	12066 cm^4
Least ,, ,, (Axis YY)	85·9 ,,	3575 ,,
Greatest Section Modulus (Axis XX)	58·9 inches^3	965 cm^3
Least ,, ,, (Axis YY)	17·5 ,,	286 ,,
Moment of Resistance (XX) at $7\frac{1}{2}$ tons stress	442 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	38 tons (7·8 ft.)	
Greatest Radius of Gyration (Axis XX)	4·22 inches	
Least ,, ,, (Axis YY)	2·30 ,,	
Maximum length rolled	about 68 feet	about 21 metres

The "MOMENT OF RESISTANCE" is the "Greatest Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: $7\frac{1}{2}$ tons per square inch).

Span (feet)	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons)	37	29	25	21	18	16	15	12	11	9	...	...	...	...
Deflection (inches)	·11	·18	·26	·35	·46	·58	·72	1·0	1·4	1·8	...	...	...	...

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about $13\frac{1}{2}$ inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·31 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet)	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons)	82	79	78	76	74	71	69	67	65	62	59	55	53	47
Nett Weight (cwts.	3·96	4·45	4·95	5·44	5·94	6·43	6·93	7·42	7·92	8·41	8·91	9·40	9·90	10·9

Weight of Stanchion Base, about 2·09 cwts.; Cap, about 0·84 cwt.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·36 tons if applied to one of the flanges, or 1·19 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

10″ $10\frac{1}{4}''$ $10\frac{1}{2}''$ 11″ $11\frac{1}{2}''$ 12″ $12\frac{1}{2}''$ $13\frac{1}{2}''$ 14″ 15″ 16″ 17″ 18″ 19″ 20″ 22″ 24″ 26″ 30″

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 120. Code Word: "ACCENT."
Nominal Dimensions 10″×10″×55 lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

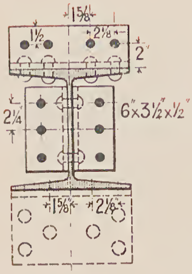


Fig. **1**

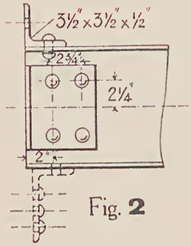


Fig. **2**

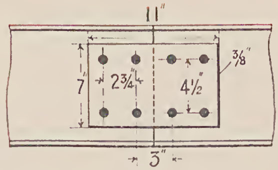


Fig. **4**

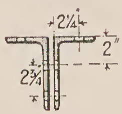


Fig. **3**

SCALE 1/16

(¾ inch = 1 foot)

Rivets and Bolts ¾″

Bolt-Holes $\frac{13}{16}$″

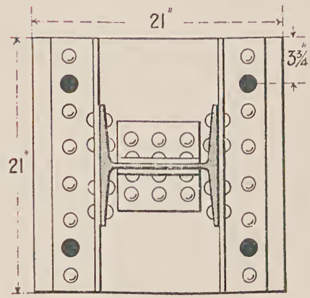


Fig. **8**

Fig. **5**

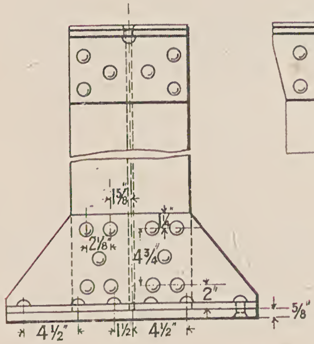


Fig. **6**

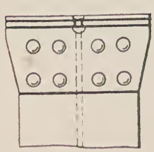
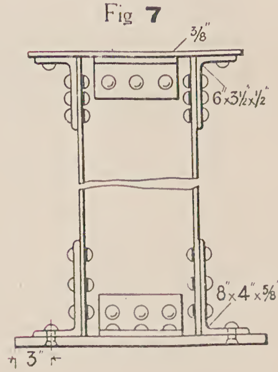


Fig **7**



Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 37·1 tons (span: 7·9 feet). The "Web Cleats" by themselves are suited to a distributed load of 21·2 tons only (span: 13·9 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, 3·06 square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 10·6 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 14·1 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 120. Code Word: "ACCENT."
Nominal Dimensions $10'' \times 10'' \times 55$ lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.	LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 40.	Broad Flange Beam(s) $10'' \times 10''$ approx. -	55 lbs. per ft.
	H. J. SKELTON & Co.'s Section No. 120	
	N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	40 ft. per ton
Figs. 1-3. Web Cleats	2 Angles $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by 7″ long - -	18 lbs. per pair
	4 $\frac{3}{4}''$ Rivets, $1\frac{1}{2}''$ grip - - - - -	2 lbs.
	6 $\frac{3}{4}''$ Bolts - - - - - - -	$5\frac{1}{2}$ lbs. say
	Gross weight of Web Cleats	$25\frac{1}{2}$ lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1 Angle $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by 10″ long - -	9 lbs.
	4 $\frac{3}{4}''$ Rivets, $1\frac{1}{4}''$ grip - - - - -	2 lbs.
	4 $\frac{3}{4}''$ Bolts - - - - - - -	4 lbs. say
	Gross weight of one Upper Flange Cleat	15 lbs.
Figs. 1, 2. Lower Flange Cleat	1 Angle $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by 10″ long - -	13 lbs.
	Gross weight including 4 Bolts & 6 Rivets	20 lbs. say
Fig. 4. Fishplates	2 $11'' \times 7'' \times \frac{3}{8}''$ - - - - - - -	$16\frac{1}{2}$ lbs. per pair
	8 $\frac{3}{4}''$ Bolts, 2″ long - - - - -	$5\frac{1}{2}$ lbs.
	Gross weight of Fishplates	22 lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2 Angles to flanges $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by 10″ long	$25\frac{1}{2}$ lbs. per pair
	2 Angles to web $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by 7″ long -	13 lbs. per pair
	1 Cover Plate (if required) $18'' \times 10'' \times \frac{3}{8}''$ -	19 lbs.
	17 $\frac{3}{4}''$ Rivets (2 countersunk) - - -	$7\frac{1}{2}$ lbs.
	(14) $\frac{3}{4}''$ Bolts - - - - - - -	$13\frac{1}{2}$ lbs. say
	Gross weight of one Light Cap (including plate)	$78\frac{1}{2}$ lbs.
Fig. 6. Stanchion Cap (heavy)	2 Angles to flanges $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by 12″ long	31 lbs. per pair
	1 Cover Plate (if required) $18'' \times 12'' \times \frac{3}{8}''$ -	23 lbs.
	21 $\frac{3}{4}''$ Rivets (2 countersunk) - - -	$9\frac{1}{2}$ lbs.
	2 Angles to web $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by 7″ long -	13 lbs. per pair
	(14) Bolts (8 of $\frac{7}{8}''$ diameter) - - - -	18 lbs. say
	Gross weight of one Heavy Cap (including plate)	$94\frac{1}{2}$ lbs.
Figs. 5-8. Stanchion Base	1 Sole Plate $21'' \times 21'' \times \frac{5}{8}''$ - - - - -	78 lbs.
	2 Angles to flanges $8'' \times 4'' \times \frac{5}{8}''$ by 21″ long -	$84\frac{1}{2}$ lbs. per pair
	2 Angles to web $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ by 7″ long -	13 lbs. per pair
	41 $\frac{3}{4}''$ Rivets (18 countersunk) - - -	18 lbs.
	4 Holding-down Bolts $1\frac{1}{4}'' \times 18''$ with $4'' \times 4'' \times \frac{3}{8}''$ plates - - - -	41 lbs.
	Gross weight of one Stanchion Base	$234\frac{1}{2}$ lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 4 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

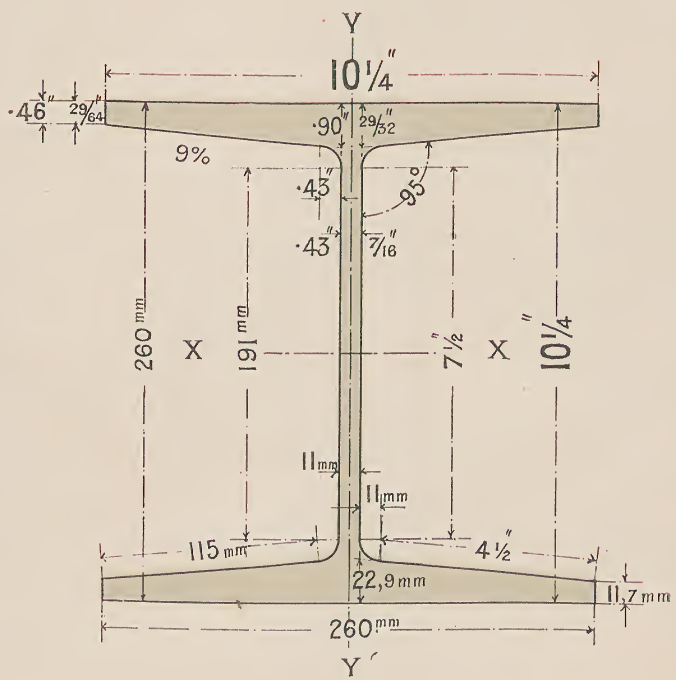
The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

For further notes, see page 19.

SECTION DRAWING OF

B.F. BEAM { Section No. 124. Code Word: "ACOLYTE."
Nominal Dimensions $10\frac{1}{4}'' \times 10\frac{1}{4}'' \times 61$ lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM { Section No. 124. Code Word: "ACOLYTE."
Nominal Dimensions $10\frac{1}{4}'' \times 10\frac{1}{4}'' \times 61$ lbs. per foot.
Actual Dimensions $10\frac{1}{4}'' \times 10\frac{1}{4}'' \times 61$ lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=36½ feet to the ton, approx.) - -	61 lbs. per foot	90·7 kilos. per m.
Sectional Area - - - - - -	17·9 sq. ins.	115·6 sq. cm.
Greatest Moment of Inertia (Axis XX) - -	345 inches^4	14352 cm^4
Least ,, ,, (Axis YY) - -	102 ,,	4261 ,,
Greatest Section Modulus (Axis XX) - -	67·4 inches^3	1104 cm^3
Least ,, ,, (Axis YY) - -	20·0 ,,	328 ,,
Moment of Resistance (xx) at 7½ tons stress -	505 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	42 tons (8 ft.)	
Greatest Radius of Gyration (Axis XX) - -	4·38 inches	
Least ,, ,, (Axis YY) - -	2·39 ,,	
Maximum length rolled - - - - -	about 68 feet	about 21 metres

The "Moment of Resistance" is the "Greatest Section Modulus" multiplied by the working stress (here taken as 7½ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: 7½ tons per square inch).

Span (feet) - -	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons) -	42	34	28	24	21	19	17	14	12	10	...	...	...	...
Deflection (inches)	·11	·17	·25	·34	·44	·56	·69	1·0	1·4	1·8	...	...	...	...

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about 13½ inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·34 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet) -	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons) -	91	88	86	84	83	80	77	75	73	70	68	65	61	55
Nett Weight (cwts.)	4·35	4·90	5·44	5·99	6·53	7·07	7·62	8·16	8·71	9·25	9·80	10·3	10·9	12·0

Weight of Stanchion Base, about 2·24 cwts.; Cap, about 0·96 cwt.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·36 tons if applied to one of the flanges, or 1·19 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 124. Code Word: "ACOLYTE."
Nominal Dimensions $10\frac{1}{4}'' \times 10\frac{1}{4}'' \times 61$ lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

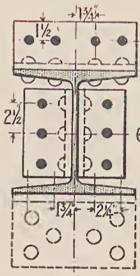


Fig. 1

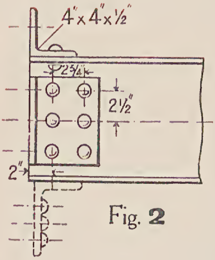


Fig. 2

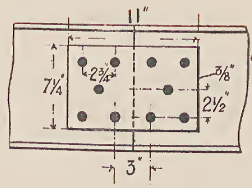


Fig. 4

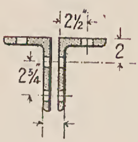


Fig. 3

SCALE 1/16

($\frac{3}{4}$ inch = 1 foot)

Rivets and Bolts $\frac{3}{4}''$

Bolt-Holes $\frac{13}{16}''$

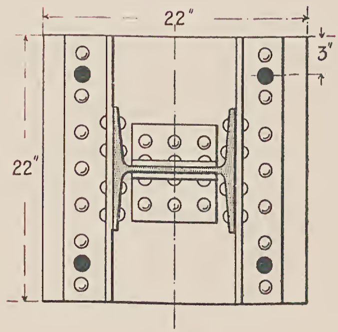


Fig. 8

Fig. 5

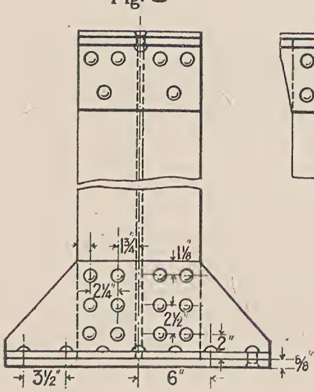


Fig. 6

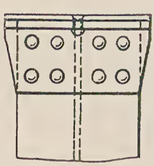
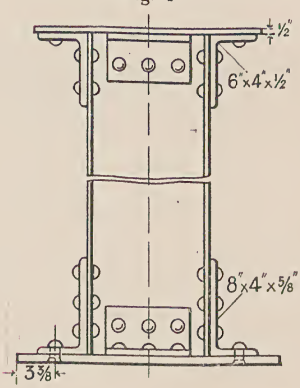


Fig. 7



Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 37·1 tons (span: 9·1 feet). The "Web Cleats" by themselves are suited to a distributed load of 21·2 tons only (span: 15·9 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, 3·36 square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 10·6 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 14·1 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 124. Code Word: "ACOLYTE."
Nominal Dimensions 10¼" × 10¼" × 61 lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.		LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 44.		Broad Flange Beam(s) 10¼" × 10¼" approx. H. J. SKELTON & Co.'s Section No. 124	61 lbs. per ft.
		N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	36½ ft. per ton
Figs. 1-3. Web Cleats	2	Angles 6" × 4" × ½" by 7¼" long	19½ lbs. per pair
	6	¾" Rivets, 1½" grip	3 lbs.
	6	¾" Bolts	5½ lbs. say
		Gross weight of Web Cleats	28 lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1	Angle 4" × 4" × ½" by 10¼" long	11 lbs.
	4	¾" Rivets, 1½" grip	2 lbs.
	4	¾" Bolts	4 lbs. say
		Gross weight of one Upper Flange Cleat	17 lbs.
Figs. 1, 2. Lower Flange Cleat	1	Angle 6" × 4" × ½" by 10¼" long	14 lbs.
		Gross weight including 4 Bolts & 6 Rivets	21 lbs. say
Fig. 4. Fishplates	2	11" × 7¼" × ⅜"	17 lbs. per pair
	10	¾" Bolts, 2½" long	7½ lbs.
		Gross weight of Fishplates	24½ lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2	Angles to flanges 6" × 4" × ½" by 10¼" long	28 lbs. per pair
	2	Angles to web 4" × 4" × ½" by 7¼" long	15½ lbs. per pair
	1	Cover Plate (if required) 19" × 10¼" × ½"	27½ lbs.
	17	¾" Rivets (2 countersunk)	7½ lbs.
	(14)	¾" Bolts	13½ lbs. say
		Gross weight of one Light Cap (including plate)	92 lbs.
Fig. 6. Stanchion Cap (heavy)	2	Angles to flanges 6" × 4" × ½" by 12" long	32½ lbs. per pair
	1	Cover Plate (if required) 19" × 12" × ½"	32½ lbs.
	21	¾" Rivets (2 countersunk)	9½ lbs.
	2	Angles to web 4" × 4" × ½" by 7¼" long	15½ lbs. per pair
	(14)	Bolts (8 of ⅞" diameter)	18 lbs. say
		Gross weight of one Heavy Cap (including plate)	108 lbs.
Figs. 5-8. Stanchion Base	1	Sole Plate 22" × 22" × ⅝"	86 lbs.
	2	Angles to flanges 8" × 4" × ⅝" by 22" long	88½ lbs. per pair
	2	Angles to web 4" × 4" × ½" by 7¼" long	15½ lbs. per pair
	47	¾" Rivets (20 countersunk)	20 lbs.
	4	Holding-down Bolts 1¼" × 18" with 4" × 4" × ⅜" plates	41 lbs.
		Gross weight of one Stanchion Base	251 lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 4 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

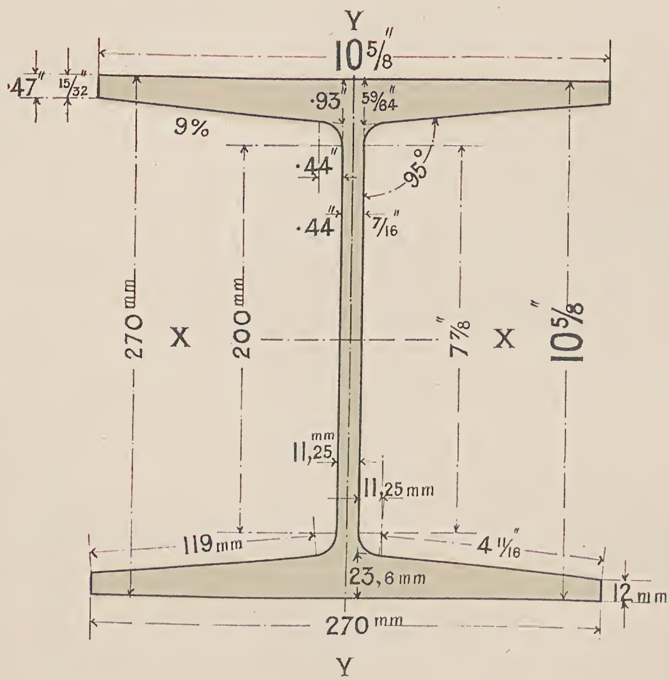
The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

For further notes, see page 19.

SECTION DRAWING OF

B.F. BEAM { Section No. 128. Code Word: "ACTOR." Nominal Dimensions $10\frac{1}{2}'' \times 10\frac{1}{2}'' \times 65$ lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM
- Section No. 128. Code Word: "ACTOR."
- Nominal Dimensions $10\frac{1}{2}'' \times 10\frac{1}{2}'' \times 65$ lbs. per foot.
- Actual Dimensions $10\frac{5}{8}'' \times 10\frac{5}{8}'' \times 65$ lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=$34\frac{1}{2}$ feet to the ton, approx.) - -	65 lbs. per foot	96·7 kilos. per m.
Sectional Area - - - - - -	19·1 sq. ins.	123·2 sq. cm.
Greatest Moment of Inertia (Axis XX) - -	397 inches^4	16529 cm^4
Least " " (Axis YY) - -	118 "	4920 "
Greatest Section Modulus (Axis XX) - -	74·7 inches^3	1224 cm^3
Least " " (Axis YY) - -	22·3 "	365 "
Moment of Resistance (XX) at $7\frac{1}{2}$ tons stress -	560 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	44 tons ($8\frac{1}{2}$ ft.)	
Greatest Radius of Gyration (Axis XX) - -	4·56 inches	
Least " " (Axis YY) - -	2·49 "	
Maximum length rolled - - - - -	about 68 feet	about 21 metres

The "MOMENT OF RESISTANCE" is the "Greatest Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: $7\frac{1}{2}$ tons per square inch).

Span (feet) - -	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons) -	*44*	37	31	27	23	21	19	16	13	12	10	...	...	...
Deflection (inches)	...	·17	·24	·32	·42	·54	·67	·96	1·3	1·7	2·2	...	...	...

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about $13\frac{1}{2}$ inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·34 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet) -	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons) -	97	95	93	91	89	87	84	81	79	77	74	71	68	61
Nett Weight (cwts.)	4·64	5·22	5·80	6·38	6·96	7·54	8·12	8·70	9·28	9·86	10·4	11·0	11·6	12·8

Weight of Stanchion Base, about 2·48 cwts.; Cap, about 0·98 cwt.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·36 tons if applied to one of the flanges, or 1·19 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

$10\frac{1}{2}''$
11"
$11\frac{1}{2}''$
12"
$12\frac{1}{2}''$
$13\frac{1}{2}''$
14"
15"
16"
17"
18"
19"
20"
22"
24"
26"
30"

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 128. Code Word: "ACTOR." Nominal Dimensions $10\frac{1}{2}'' \times 10\frac{1}{2}'' \times 65$ lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

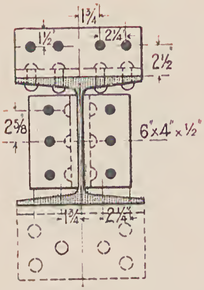


Fig. **1**

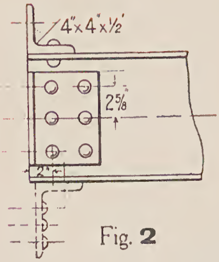


Fig. **2**

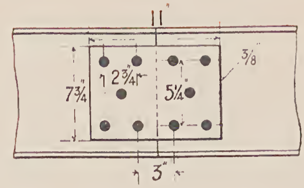


Fig. **4**

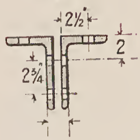


Fig. **3**

SCALE 1/16

($\frac{3}{4}$ inch = 1 foot)

Rivets and Bolts $\frac{3}{4}''$

Bolt-Holes $\frac{13}{16}''$

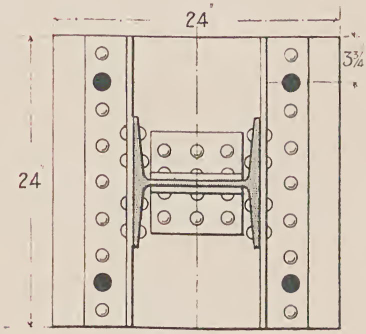


Fig. **8**

Fig. **5**

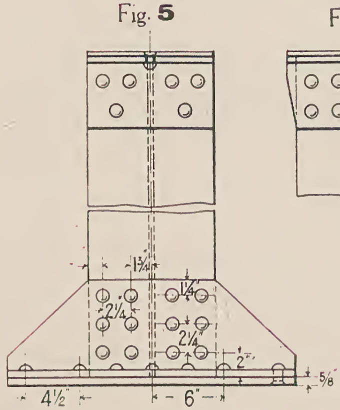


Fig. **6**

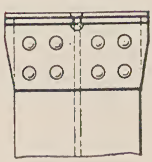
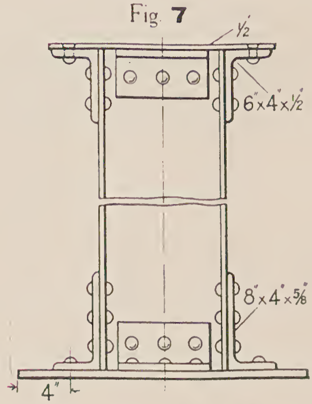


Fig. **7**



Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 37·1 tons (span: 10·1 feet). The "Web Cleats" by themselves are suited to a distributed load of 21·2 tons only (span: 17·6 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, 4 square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 10·6 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 14·1 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 128. Code Word : "ACTOR."
Nominal Dimensions $10\frac{1}{2}'' \times 10\frac{1}{2}'' \times 65$ lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.	LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 48.	Broad Flange Beam(s) $10\frac{1}{2}'' \times 10\frac{1}{2}''$ approx.	65 lbs. per ft.
	H. J. SKELTON & Co.'s Section No. 128	
	N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	$34\frac{1}{2}$ ft. per ton
Figs. 1-3. Web Cleats	2 Angles $6'' \times 4'' \times \frac{1}{2}''$ by $7\frac{3}{4}''$ long	21 lbs. per pair
	6 $\frac{3}{4}''$ Rivets, $1\frac{1}{2}''$ grip	3 lbs.
	6 $\frac{3}{4}''$ Bolts	$5\frac{1}{2}$ lbs. say
	Gross weight of Web Cleats	$29\frac{1}{2}$ lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1 Angle $4'' \times 4'' \times \frac{1}{2}''$ by $10\frac{1}{2}''$ long	11 lbs.
	4 $\frac{3}{4}''$ Rivets, $1\frac{3}{8}''$ grip	2 lbs.
	4 $\frac{3}{4}''$ Bolts	4 lbs. say
	Gross weight of one Upper Flange Cleat	17 lbs.
Figs. 1, 2. Lower Flange Cleat	1 Angle $6'' \times 4'' \times \frac{1}{2}''$ by $10\frac{1}{2}''$ long	14 lbs.
	Gross weight including 4 Bolts & 6 Rivets	21 lbs. say
Fig. 4. Fishplates	2 $11'' \times 7\frac{3}{4}'' \times \frac{3}{8}''$	18 lbs. per pair
	10 $\frac{3}{4}''$ Bolts, $2\frac{1}{2}''$ long	$7\frac{1}{2}$ lbs.
	Gross weight of Fishplates	$25\frac{1}{2}$ lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2 Angles to flanges $6'' \times 4'' \times \frac{1}{2}''$ by $10\frac{1}{2}''$ long	28 lbs. per pair
	2 Angles to web $4'' \times 4'' \times \frac{1}{2}''$ by $7\frac{3}{4}''$ long	$16\frac{1}{2}$ lbs. per pair
	1 Cover Plate (if required) $19'' \times 10\frac{1}{2}'' \times \frac{1}{2}''$	$28\frac{1}{2}$ lbs.
	17 $\frac{3}{4}''$ Rivets (2 countersunk)	8 lbs.
	(14) $\frac{3}{4}''$ Bolts	$13\frac{1}{2}$ lbs. say
	Gross weight of one Light Cap (including plate)	$94\frac{1}{2}$ lbs.
Fig. 6. Stanchion Cap (heavy)	2 Angles to flanges $6'' \times 4'' \times \frac{1}{2}''$ by $12''$ long	$32\frac{1}{2}$ lbs. per pair
	1 Cover Plate (if required) $19'' \times 12'' \times \frac{1}{2}''$	$32\frac{1}{2}$ lbs.
	21 $\frac{3}{4}''$ Rivets (2 countersunk)	10 lbs.
	2 Angles to web $4'' \times 4'' \times \frac{1}{2}''$ by $7\frac{3}{4}''$ long	$16\frac{1}{2}$ lbs. per pair
	(14) Bolts (8 of $\frac{7}{8}''$ diameter)	18 lbs. say
	Gross weight of one Heavy Cap (including plate)	$109\frac{1}{2}$ lbs.
Figs. 5-8. Stanchion Base	1 Sole Plate $24'' \times 24'' \times \frac{5}{8}''$	102 lbs.
	2 Angles to flanges $8'' \times 4'' \times \frac{5}{8}''$ by $24''$ long	97 lbs. per pair
	2 Angles to web $4'' \times 4'' \times \frac{1}{2}''$ by $7\frac{3}{4}''$ long	$16\frac{1}{2}$ lbs. per pair
	47 $\frac{3}{4}''$ Rivets (20 countersunk)	21 lbs.
	4 Holding-down Bolts $1\frac{1}{4}'' \times 18''$ with $4'' \times 4'' \times \frac{3}{8}''$ plates	41 lbs.
	Gross weight of one Stanchion Base	$277\frac{1}{2}$ lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 4 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

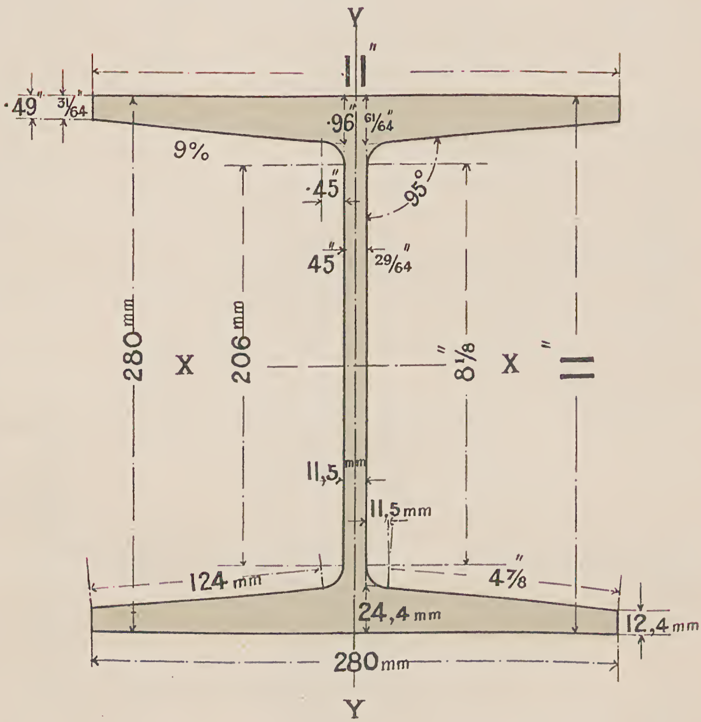
For further notes, see page 19.

SECTION DRAWING OF

B.F. BEAM { Section No. 132. Code Word: "ACTUARY."
Nominal Dimensions 11″×11″×70 lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]

STOCK SIZE.



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM
- Section No. 132. Code Word: "ACTUARY."
- Nominal Dimensions 11″×11″×70 lbs. per foot.
- Actual Dimensions 11″×11″×70 lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=32 feet to the ton, approx.) - -	70 lbs. per foot	103·4 kilos. per m.
Sectional Area - - - - - -	20·4 sq. ins.	131·8 sq. cm.
Greatest Moment of Inertia (Axis xx) - -	458 inches^4	19052 cm^4
Least ,, ,, (Axis yy) - -	136 ,,	5671 ,,
Greatest Section Modulus (Axis xx) - -	83·1 inches^3	1361 cm^3
Least ,, ,, (Axis yy) - -	24·7 ,,	405 ,,
Moment of Resistance (xx) at 7½ tons stress -	623 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	47 tons (8·8 ft.)	
Greatest Radius of Gyration (Axis xx) - -	4·73 inches	
Least ,, ,, (Axis yy) - -	2·58 ,,	
Maximum length rolled - - - - -	about 68 feet	about 21 metres

The "Moment of Resistance" is the "Greatest Section Modulus" multiplied by the working stress (here taken as 7½ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: 7½ tons per square inch).

Span (feet) - -	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons) -	*47*	42	35	30	26	23	21	17	15	13	11	...	...	...
Deflection (inches)	...	·16	·23	·31	·41	·52	·64	·92	1·3	1·6	2·1	...	...	...

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about 18 inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·36 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet) -	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons) -	104	103	100	97	96	94	91	88	86	84	81	78	75	68
Nett Weight (cwts.)	4·96	5·58	6·20	6·82	7·44	8·06	8·68	9·30	9·92	10·5	11·2	11·8	12·4	13·6

Weight of Stanchion Base, about 2½ cwts.; Cap, about 1 cwt.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·36 tons if applied to one of the flanges, or 1·19 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 132. Code Word: "ACTUARY." Nominal Dimensions 11″ × 11″ × 70 lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

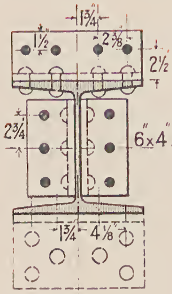


Fig. 1

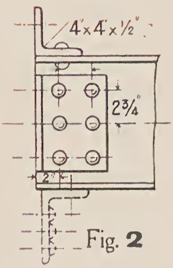


Fig. 2

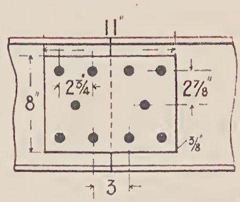


Fig. 4

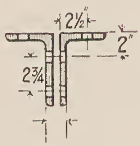


Fig. 3

SCALE 1/16

(¾ inch = 1 foot)

Rivets and Bolts ¾″

Bolt-Holes $\frac{13}{16}$″

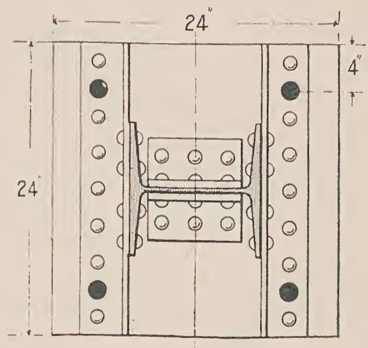


Fig. 8

Fig. 5

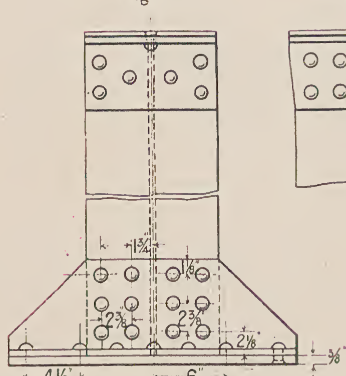


Fig. 6

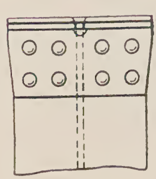
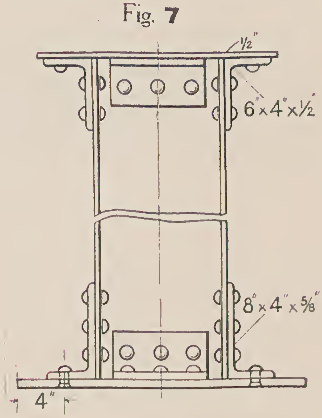


Fig. 7



Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 37·1 tons (span: 11·2 feet). The "Web Cleats" by themselves are suited to a distributed load of 21·2 tons only (span: 19·6 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, 4 square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 10·6 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 14·1 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 132. Code Word: "ACTUARY." Nominal Dimensions $11'' \times 11'' \times 70$ lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.	LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 52.	Broad Flange Beam(s) $11'' \times 11''$ approx. - H. J. SKELTON & Co.'s Section No. 132	70 lbs. per ft.
	N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	32 ft. per ton
Figs. 1-3. Web Cleats	2 Angles $6'' \times 4'' \times \frac{1}{2}''$ by 8″ long - - -	$21\frac{1}{2}$ lbs. per pair
	6 $\frac{3}{4}''$ Rivets, $1\frac{1}{2}''$ grip - - - - -	3 lbs.
	6 $\frac{3}{4}''$ Bolts - - - - - - -	$5\frac{1}{2}$ lbs. say
	Gross weight of Web Cleats	30 lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1 Angle $4'' \times 4'' \times \frac{1}{2}''$ by 11″ long - - -	12 lbs.
	4 $\frac{3}{4}''$ Rivets, $1\frac{3}{8}''$ grip - - - - -	2 lbs.
	4 $\frac{3}{4}''$ Bolts - - - - - - -	4 lbs. say
	Gross weight of one Upper Flange Cleat	18 lbs.
Figs. 1, 2. Lower Flange Cleat	1 Angle $6'' \times 4'' \times \frac{1}{2}''$ by 11″ long - - -	15 lbs.
	Gross weight including 4 Bolts & 6 Rivets	22 lbs. say
Fig. 4. Fishplates	2 $11'' \times 8'' \times \frac{3}{8}''$ - - - - - - -	19 lbs. per pair
	10 $\frac{3}{4}''$ Bolts, $2\frac{1}{4}''$ long - - - - -	$7\frac{1}{2}$ lbs.
	Gross weight of Fishplates	$26\frac{1}{2}$ lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2 Angles to flanges $6'' \times 4'' \times \frac{1}{2}''$ by 11″ long -	30 lbs. per pair
	2 Angles to web $4'' \times 4'' \times \frac{1}{2}''$ by 8″ long -	17 lbs. per pair
	1 Cover Plate (if required) $20'' \times 11'' \times \frac{1}{2}''$ -	31 lbs.
	17 $\frac{3}{4}''$ Rivets (2 countersunk) - - - -	8 lbs.
	(14) $\frac{3}{4}''$ Bolts - - - - - - -	$13\frac{1}{2}$ lbs. say
	Gross weight of one Light Cap (including plate)	$99\frac{1}{2}$ lbs.
Fig. 6. Stanchion Cap (heavy)	2 Angles to flanges $6'' \times 4'' \times \frac{1}{2}''$ by 12″ long -	$32\frac{1}{2}$ lbs. per pair
	1 Cover Plate (if required) $20'' \times 12'' \times \frac{1}{2}''$ -	34 lbs.
	2 Angles to web $4'' \times 4'' \times \frac{1}{2}''$ by 8″ long -	17 lbs. per pair
	21 $\frac{3}{4}''$ Rivets (2 countersunk) - - - -	10 lbs.
	(14) Bolts (4 of $\frac{7}{8}''$ diameter) - - - -	18 lbs. say
	Gross weight of one Heavy Cap (including plate)	$111\frac{1}{2}$ lbs.
Figs. 5-8. Stanchion Base	1 Sole Plate $24'' \times 24'' \times \frac{5}{8}''$ - - - -	102 lbs.
	2 Angles to flanges $8'' \times 4'' \times \frac{5}{8}''$ by 24″ long -	97 lbs. per pair
	2 Angles to web $4'' \times 4'' \times \frac{1}{2}''$ by 8″ long -	17 lbs. per pair
	(47) $\frac{3}{4}''$ Rivets (20 countersunk) - - - -	21 lbs.
	4 Holding-down Bolts $1\frac{1}{4}'' \times 18''$ with $4\frac{1}{2}'' \times 4\frac{1}{2}'' \times \frac{3}{8}''$ plates - - - -	43 lbs.
	Gross weight of one Stanchion Base	280 lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 4 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

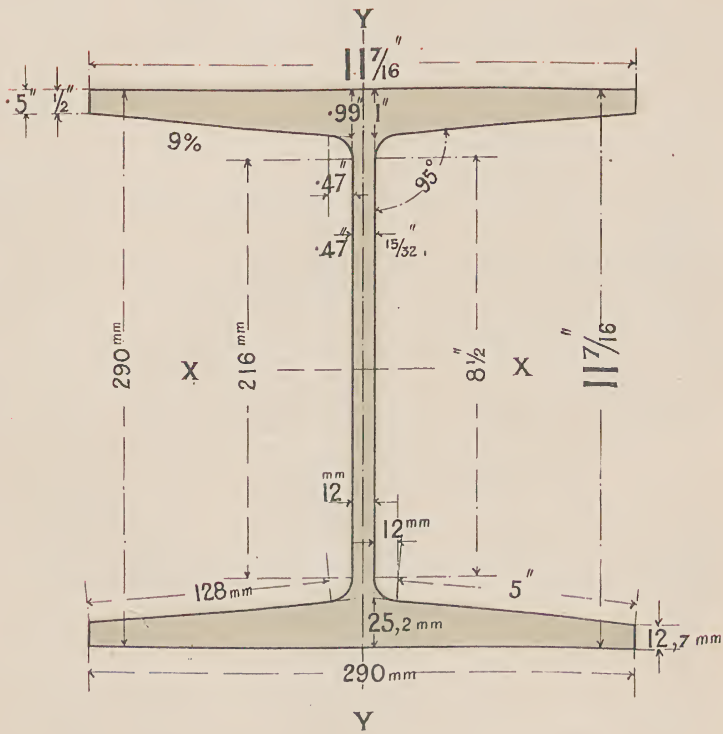
The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

For further notes, see page 19.

SECTION DRAWING OF

B.F. BEAM { Section No. 136. Code Word: "ADAMANT."
Nominal Dimensions $11\frac{1}{2}'' \times 11\frac{1}{2}'' \times 75$ lbs. per foot.

[For Properties, see opposite page.   For Drawings of Connections, P.T.O.]



SCALE 1/4   (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM
- Section No. 136. Code Word: "ADAMANT."
- Nominal Dimensions $11\frac{1}{2}'' \times 11\frac{1}{2}'' \times 75$ lbs. per foot.
- Actual Dimensions $11\frac{7}{16}'' \times 11\frac{7}{16}'' \times 75$ lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=30 feet to the ton, approx.)	75 lbs. per foot	110·8 kilos. per m.
Sectional Area	21·9 sq. ins.	141·1 sq. cm.
Greatest Moment of Inertia (Axis XX)	526 inches^4	21866 cm^4
Least „ „ (Axis YY)	154 „	6417 „
Greatest Section Modulus (Axis XX)	92·0 inches^3	1508 cm^3
Least „ „ (Axis YY)	27·0 „	443 „
Moment of Resistance (XX) at $7\frac{1}{2}$ tons stress	690 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	51 tons (9 ft.)	
Greatest Radius of Gyration (Axis XX)	4·90 inches	
Least „ „ (Axis YY)	2·66 „	
Maximum length rolled	about 68 feet	about 21 metres

The "MOMENT OF RESISTANCE" is the "Greatest Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: $7\frac{1}{2}$ tons per square inch).

Span (feet)	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons)	*51*	46	38	33	29	25	23	19	16	14	13	11	...	...
Deflection (inches)	...	·15	·22	·30	·40	·50	·62	·90	1·2	1·6	2·0	2·5	...	...

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about 18 inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·37 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet)	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons)	111	110	107	105	103	101	99	96	93	91	88	85	83	75
Nett Weight (cwts.)	5·31	5·98	6·64	7·30	7·97	8·63	9·30	9·96	10·6	11·3	12·0	12·6	13·3	14·6

Weight of Stanchion Base, about 2·40 cwts.; Cap, about 0·99 cwt.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·36 tons if applied to one of the flanges, or 1·19 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 136. Code Word: "ADAMANT."
Nominal Dimensions $11\frac{1}{2}'' \times 11\frac{1}{2}'' \times 75$ lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

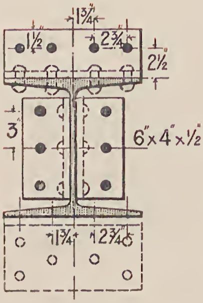


Fig. 1

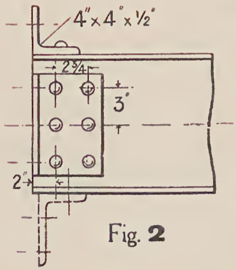


Fig. 2

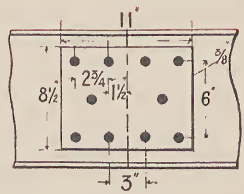


Fig 4

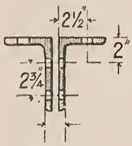


Fig. 3

SCALE 1/16

($\frac{3}{4}$ inch = 1 foot)

Rivets and Bolts $\frac{3}{4}''$

Bolt-Holes $\frac{13}{16}''$

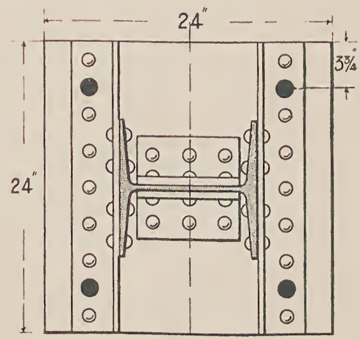


Fig. 8

Fig. 5

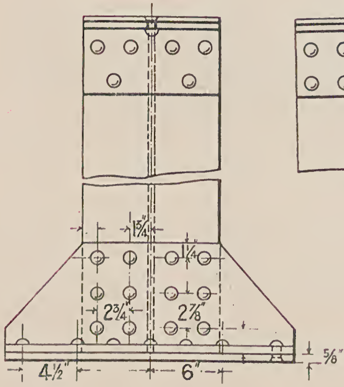


Fig. 6

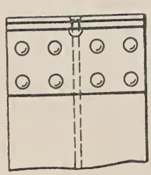
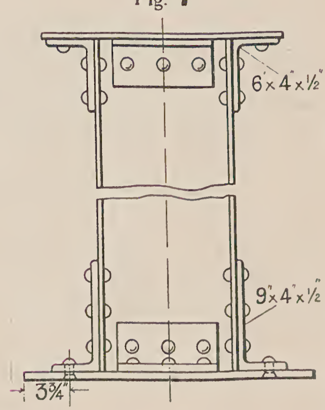


Fig. 7



Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 37·1 tons (span: 12·4 feet). The "Web Cleats" by themselves are suited to a distributed load of 21·2 tons only (span: 21·7 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, 4 square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 10·6 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 14·1 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 136. Code Word: "ADAMANT."
Nominal Dimensions $11\frac{1}{2}'' \times 11\frac{1}{2}'' \times 75$ lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.	LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 56.	Broad Flange Beam(s) $11\frac{1}{2}'' \times 11\frac{1}{2}''$ approx. H. J. SKELTON & Co.'s Section No. 136	75 lbs. per ft.
	N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	30 ft. per ton
Figs. 1-3. Web Cleats	2 Angles $6'' \times 4'' \times \frac{1}{2}''$ by $8\frac{1}{2}''$ long	23 lbs. per pair
	6 $\frac{3}{4}''$ Rivets, $1\frac{1}{2}''$ grip	3 lbs.
	6 $\frac{3}{4}''$ Bolts	$5\frac{1}{2}$ lbs. say
	Gross weight of Web Cleats	$31\frac{1}{2}$ lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1 Angle $4'' \times 4'' \times \frac{1}{2}''$ by $11\frac{1}{2}''$ long	$12\frac{1}{2}$ lbs.
	4 $\frac{3}{4}''$ Rivets, $1\frac{3}{8}''$ grip	2 lbs.
	4 $\frac{3}{4}''$ Bolts	4 lbs. say
	Gross weight of one Upper Flange Cleat	$18\frac{1}{2}$ lbs.
Figs. 1, 2. Lower Flange Cleat	1 Angle $6'' \times 4'' \times \frac{1}{2}''$ by $11\frac{1}{2}''$ long	$15\frac{1}{2}$ lbs.
	Gross weight including 4 Bolts & 6 Rivets	$22\frac{1}{2}$ lbs. say
Fig. 4. Fishplates	2 $11'' \times 8\frac{1}{2}'' \times \frac{3}{8}''$	20 lbs. per pair
	10 $\frac{3}{4}''$ Bolts, $2\frac{1}{2}''$ long	$7\frac{1}{2}$ lbs.
	Gross weight of Fishplates	$27\frac{1}{2}$ lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2 Angles to flanges $6'' \times 4'' \times \frac{1}{2}''$ by $11\frac{1}{2}''$ long	31 lbs. per pair
	2 Angles to web $4'' \times 4'' \times \frac{1}{2}''$ by $8\frac{1}{2}''$ long	18 lbs. per pair
	1 Cover Plate (if required) $20'' \times 11\frac{1}{2}'' \times \frac{1}{2}''$	33 lbs.
	17 $\frac{3}{4}''$ Rivets (2 countersunk)	8 lbs.
	(14) $\frac{3}{4}''$ Bolts	$14\frac{1}{2}$ lbs. say
	Gross weight of one Light Cap (including plate)	$104\frac{1}{2}$ lbs.
Fig. 6. Stanchion Cap (heavy)	Angles and Cover Plate as for Light Cap	82 lbs.
	21 $\frac{3}{4}''$ Rivets (2 countersunk)	10 lbs.
	(14) Bolts (8 of $\frac{7}{8}''$ diameter)	19 lbs. say
	Gross weight of one Heavy Cap (including plate)	111 lbs.
Figs. 5-8. Stanchion Base	1 Sole Plate $24'' \times 24'' \times \frac{5}{8}''$	102 lbs.
	2 Angles to flanges $9'' \times 4'' \times \frac{1}{2}''$ by $24''$ long	85 lbs. per pair
	2 Angles to web $4'' \times 4'' \times \frac{1}{2}''$ by $8\frac{1}{2}''$ long	18 lbs. per pair
	47 $\frac{3}{4}''$ Rivets (20 countersunk)	21 lbs.
	4 Holding-down Bolts $1\frac{1}{4}'' \times 18''$ with $4\frac{1}{2}'' \times 4\frac{1}{2}'' \times \frac{3}{8}''$ plates	43 lbs.
	Gross weight of one Stanchion Base	269 lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 4 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

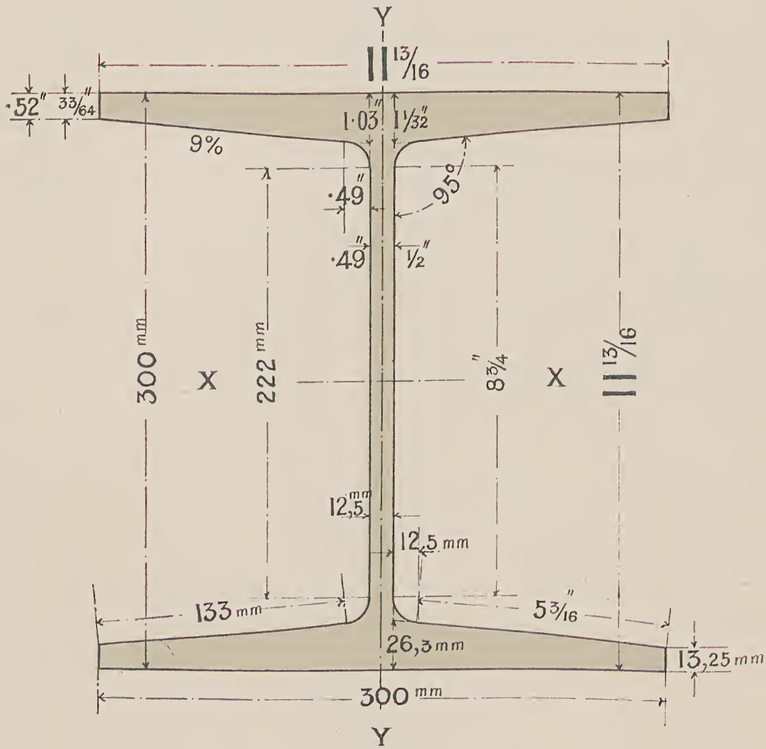
For further notes, see page 19.

SECTION DRAWING OF

B.F. BEAM { Section No. 140. Code Word: "ADDER."
Nominal Dimensions 12″×12″×80 lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]

STOCK SIZE.



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM
- Section No. 140. Code Word: "ADDER."
- Nominal Dimensions $12'' \times 12'' \times 80$ lbs. per foot.
- Actual Dimensions $11\frac{13}{16}'' \times 11\frac{13}{16}'' \times 80$ lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=28 feet to the ton, approx.) - -	80 lbs. per foot	119·4 kilos. per m.
Sectional Area - - - - - -	23·6 sq. ins.	152·1 sq. cm.
Greatest Moment of Inertia (Axis XX) - -	605 inches^4	25201 cm^4
Least ,, ,, (Axis YY) - -	180 ,,	7494 ,,
Greatest Section Modulus (Axis XX) - -	103 inches^3	1680 cm^3
Least ,, ,, (Axis YY) - -	30·5 ,,	500 ,,
Moment of Resistance (XX) at $7\frac{1}{2}$ tons stress -	769 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	55 tons (9·4 ft.)	
Greatest Radius of Gyration (Axis XX) - -	5·07 inches	
Least ,, ,, (Axis YY) - -	2·76 ,,	
Maximum length rolled - - - - -	about 68 feet	about 21 metres

The "MOMENT OF RESISTANCE" is the "Greatest Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: $7\frac{1}{2}$ tons per square inch).

Span (feet) - -	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons) -	*55*	51	43	37	32	28	26	21	18	16	14	13	...	...
Deflection (inches)	...	·15	·22	·29	·39	·49	·60	·86	1·2	1·5	1·9	2·4	...	...

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about 18 inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·49 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet) -	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons) -	120	119	117	114	112	110	108	105	102	99	97	94	91	85
Nett Weight (cwts.)	5·73	6·44	7·16	7·88	8·59	9·31	10·0	10·7	11·5	12·2	12·9	13·6	14·3	15·8

Weight of Stanchion Base, about 2·91 cwts.; Cap, about 1·19 cwts.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·36 tons if applied to one of the flanges, or 1·19 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 140. Code Word: "ADDER."
Nominal Dimensions 12″×12″×80 lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

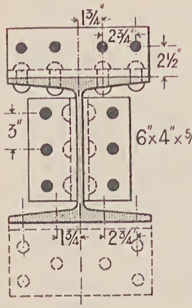


Fig. **1**

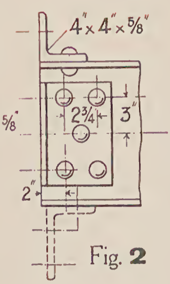


Fig. **2**

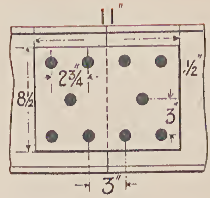


Fig. **4**

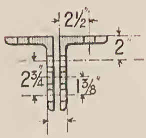


Fig. **3**

SCALE 1/16

($\frac{3}{4}$ inch = 1 foot)

Rivets and Bolts $\frac{7}{8}$″

Bolt-Holes $\frac{15}{16}$″

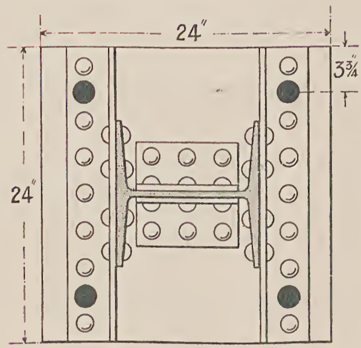


Fig. **8**

Fig. **5**

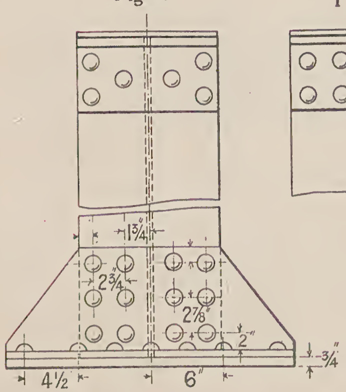


Fig. **6**

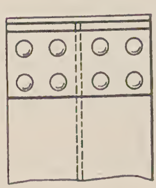
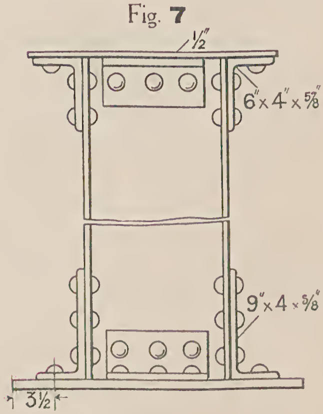


Fig. **7**



Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 50·5 tons (span: 10·1 feet). The "Web Cleats" by themselves are suited to a distributed load of 28·9 tons only (span: 17·8 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, 4 square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 14·4 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 19·2 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 140. Code Word: "ADDER."
Nominal Dimensions $12'' \times 12'' \times 80$ lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.		LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 60.		Broad Flange Beam(s) $12'' \times 12''$ approx. - H. J. SKELTON & Co.'s Section No. 140	80 lbs. per ft.
		N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	28 ft. per ton
Figs. 1-3. Web Cleats	2	Angles $6'' \times 4'' \times \frac{5}{8}''$ by $8\frac{1}{2}''$ long - -	28 lbs. per pair
	5	$\frac{7}{8}''$ Rivets, $1\frac{3}{4}''$ grip - - - - -	4 lbs.
	6	$\frac{7}{8}''$ Bolts - - - - - - -	9 lbs. say
		Gross weight of Web Cleats	41 lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1	Angle $4'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long - -	$15\frac{1}{2}$ lbs.
	4	$\frac{7}{8}''$ Rivets, $1\frac{5}{8}''$ grip - - - - -	3 lbs.
	4	$\frac{7}{8}''$ Bolts - - - - - - -	6 lbs. say
		Gross weight of one Upper Flange Cleat	$24\frac{1}{2}$ lbs.
Figs. 1, 2. Lower Flange Cleat	1	Angle $6'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long - -	$19\frac{1}{2}$ lbs.
		Gross weight including 4 Bolts & 6 Rivets	30 lbs. say
Fig. 4. Fishplates	2	$11'' \times 8\frac{1}{2}'' \times \frac{1}{2}''$ - - - - - - -	$26\frac{1}{2}$ lbs. per pair
	10	$\frac{7}{8}''$ Bolts, $2\frac{1}{2}''$ long - - - - -	$11\frac{1}{2}$ lbs.
		Gross weight of Fishplates	38 lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2	Angles to flanges $6'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long	39 lbs. per pair
	2	Angles to web $4'' \times 4'' \times \frac{5}{8}''$ by $8\frac{1}{2}''$ long -	$22\frac{1}{2}$ lbs. per pair
	1	Cover plate (if required) $20'' \times 12'' \times \frac{1}{2}''$ -	34 lbs.
	17	$\frac{7}{8}''$ Rivets (2 countersunk) - - -	13 lbs.
	(14)	$\frac{7}{8}''$ Bolts - - - - - - -	22 lbs. say
		Gross weight of one Light Cap (including plate)	$130\frac{1}{2}$ lbs.
Fig. 6. Stanchion Cap (heavy)		Same as for Light Cap, but 4 extra Rivets - - - - - -	3 lbs. extra
		Gross weight of one Heavy Cap (including plate)	$133\frac{1}{2}$ lbs.
Figs. 5-8. Stanchion Base	1	Sole Plate $24'' \times 24'' \times \frac{3}{4}''$ - - - - -	122 lbs.
	2	Angles to flanges $9'' \times 4'' \times \frac{5}{8}''$ by $24''$ long	105 lbs. per pair
	2	Angles to web $4'' \times 4'' \times \frac{5}{8}''$ by $8\frac{1}{2}''$ long -	$22\frac{1}{2}$ lbs. per pair
	47	$\frac{7}{8}''$ Rivets (20 countersunk) - - -	33 lbs.
	4	Holding-down Bolts $1\frac{1}{4}'' \times 18''$ with $4\frac{1}{2}'' \times 4\frac{1}{2}'' \times \frac{3}{8}''$ plates - - - -	43 lbs.
		Gross weight of one Stanchion Base	$325\frac{1}{2}$ lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 4 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

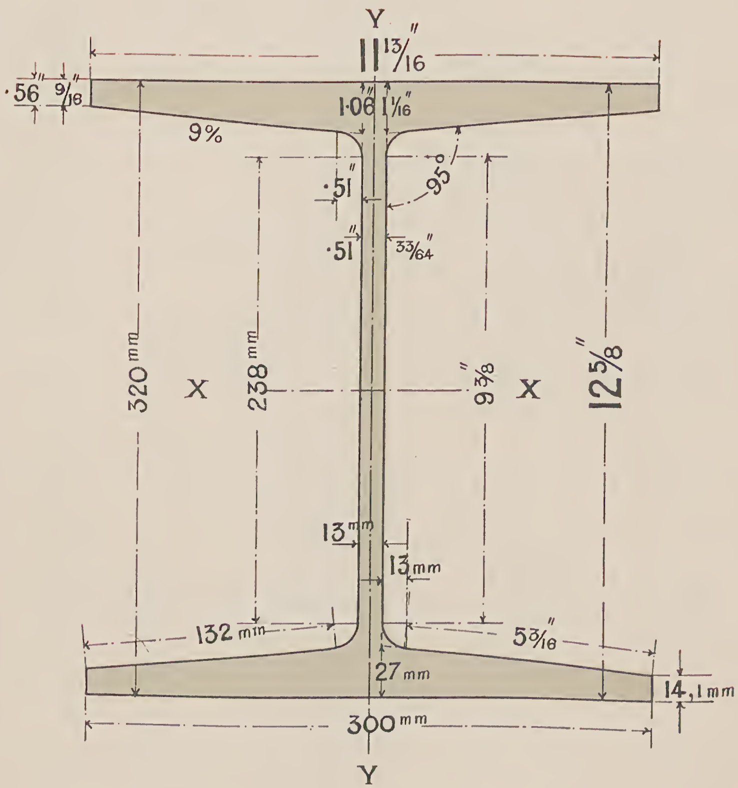
The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

For further notes, see page 19.

SECTION DRAWING OF

B.F. BEAM { Section No. 144. Code Word: "ADEPT."
Nominal Dimensions $12\frac{1}{2}'' \times 12'' \times 85$ lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM
- Section No. 144. Code Word: "ADEPT."
- Nominal Dimensions $12\frac{1}{2}'' \times 12'' \times 85$ lbs. per foot.
- Actual Dimensions $12\frac{5}{8}'' \times 11\frac{13}{16}'' \times 85$ lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=26 feet to the ton, approx.)	85 lbs. per foot	126·2 kilos. per m.
Sectional Area	24·9 sq. ins.	160·7 sq. cm.
Greatest Moment of Inertia (Axis XX)	724 inches^4	30119 cm^4
Least ,, ,, (Axis YY)	189 ,,	7867 ,,
Greatest Section Modulus (Axis XX)	115 inches^3	1882 cm^3
Least ,, ,, (Axis YY)	32·0 ,,	524 ,,
Moment of Resistance (XX) at $7\frac{1}{2}$ tons stress	862 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	59 tons (9·7 ft.)	
Greatest Radius of Gyration (Axis XX)	5·39 inches	
Least ,, ,, (Axis YY)	2·75 ,,	
Maximum length rolled	about 68 feet	about 21 metres

The "Moment of Resistance" is the "Greatest Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: $7\frac{1}{2}$ tons per square inch).

Span (feet)	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons)	*59*	58	48	41	36	32	29	24	21	18	16	14	...	...
Deflection (inches)	...	·14	·20	·27	·36	·45	·56	·81	1·1	1·4	1·8	2·2	...	...

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about 18 inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·49 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet)	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons)	128	125	123	120	118	116	114	111	108	105	102	99	96	89
Nett Weight (cwts.)	6·05	6·81	7·56	8·32	9·08	9·83	10·6	11·3	12·1	12·9	13·6	14·4	15·1	16·6

Weight of Stanchion Base, about 3·11 cwts.; Cap, about 1·22 cwts.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·37 tons if applied to one of the flanges, or 1·20 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 144. Code Word: "ADEPT."
Nominal Dimensions $12\frac{1}{2}'' \times 12'' \times 85$ lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

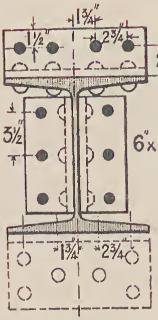


Fig. 1

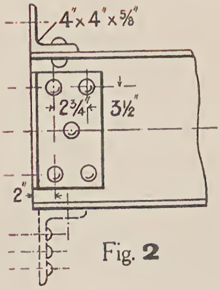


Fig. 2

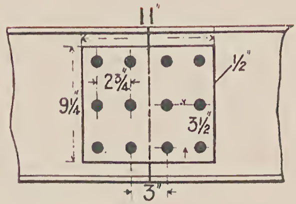


Fig. 4

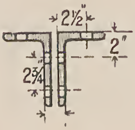


Fig. 3

SCALE 1/16

($\frac{3}{4}$ inch = 1 foot)

Rivets and Bolts $\frac{7}{8}''$

Bolt-Holes $\frac{15}{16}''$

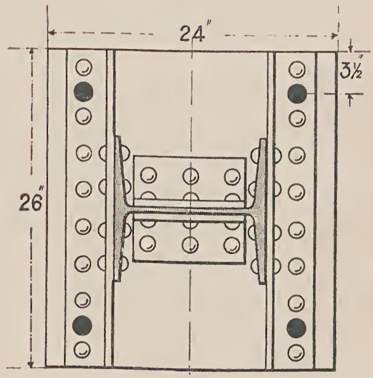


Fig. 8

Fig. 5

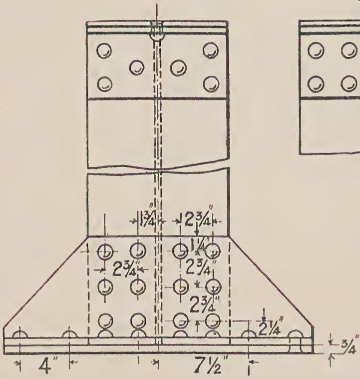


Fig. 6

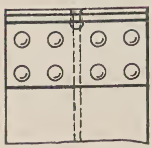
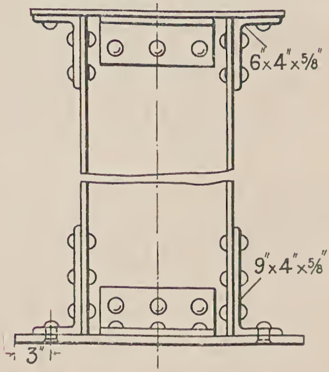


Fig. 7



Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 50·5 tons (span: 11·4 feet). The "Web Cleats" by themselves are suited to a distributed load of 28·9 tons only (span: 19·9 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, $4\frac{1}{3}$ square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 14·4 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 19·2 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 144. Code Word: "ADEPT."
Nominal Dimensions $12\frac{1}{2}'' \times 12'' \times 85$ lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.	LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 64.	Broad Flange Beam(s) $12\frac{1}{2}'' \times 12''$ approx. H. J. SKELTON & Co.'s Section No. 144	85 lbs. per ft.
	N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	26 ft. per ton
Figs. 1-3. Web Cleats	2 Angles $6'' \times 4'' \times \frac{5}{8}''$ by $9\frac{1}{4}''$ long - -	$30\frac{1}{2}$ lbs. per pair
	5 $\frac{7}{8}''$ Rivets, $1\frac{3}{4}''$ grip - - - - -	4 lbs.
	6 $\frac{7}{8}''$ Bolts - - - - - - -	9 lbs. say
	Gross weight of Web Cleats	$43\frac{1}{2}$ lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1 Angle $4'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long - -	$15\frac{1}{2}$ lbs.
	4 $\frac{7}{8}''$ Rivets, $1\frac{5}{8}''$ grip - - - - -	3 lbs.
	4 $\frac{7}{8}''$ Bolts - - - - - - -	6 lbs. say
	Gross weight of one Upper Flange Cleat	$24\frac{1}{2}$ lbs.
Figs. 1, 2. Lower Flange Cleat	1 Angle $6'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long - -	$19\frac{1}{2}$ lbs.
	Gross weight including 4 Bolts & 6 Rivets	30 lbs. say
Fig. 4. Fishplates	2 $11'' \times 9\frac{1}{4}'' \times \frac{1}{2}''$ - - - - - - -	29 lbs. per pair
	12 $\frac{7}{8}''$ Bolts, $2\frac{1}{2}''$ long - - - - -	14 lbs.
	Gross weight of Fishplates	43 lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2 Angles to flanges $6'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long	39 lbs. per pair
	2 Angles to web $4'' \times 4'' \times \frac{5}{8}''$ by $9\frac{1}{4}''$ long -	24 lbs. per pair
	1 Cover Plate (if required) $21'' \times 12'' \times \frac{1}{2}''$ -	36 lbs.
	17 $\frac{7}{8}''$ Rivets (2 countersunk) - - -	13 lbs.
	(14) $\frac{7}{8}''$ Bolts - - - - - - -	22 lbs. say
	Gross weight of one Light Cap (including plate)	134 lbs.
Fig. 6. Stanchion Cap (heavy)	Same as for Light Cap, but 4 extra Rivets - - - - - -	3 lbs. extra
	Gross weight of one Heavy Cap (including plate)	137 lbs.
Figs. 5-8. Stanchion Base	1 Sole Plate $24'' \times 26'' \times \frac{3}{4}''$ - - - - -	133 lbs.
	2 Angles to flanges $9'' \times 4'' \times \frac{5}{8}''$ by $26''$ long	114 lbs. per pair
	2 Angles to web $4'' \times 4'' \times \frac{5}{8}''$ by $9\frac{1}{4}''$ long -	24 lbs. per pair
	49 $\frac{7}{8}''$ Rivets (22 countersunk) - - -	34 lbs.
	4 Holding-down Bolts $1\frac{1}{4}'' \times 18''$ with $4\frac{1}{2}'' \times 4\frac{1}{2}'' \times \frac{3}{8}''$ plates - - - -	43 lbs.
	Gross weight of one Stanchion Base	348 lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 4 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

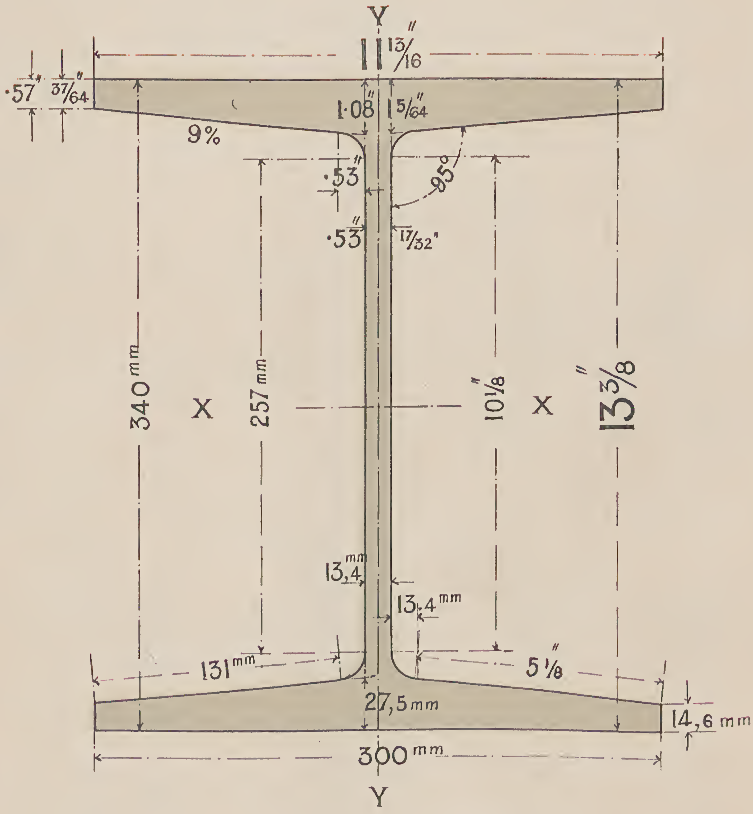
The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges

For further notes, see page 19.

SECTION DRAWING OF

B.F. BEAM { Section No. 148. Code Word: "ADMIRER." Nominal Dimensions 13½" × 12" × 88 lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM Section No. 148. Code Word: "ADMIRER."
Nominal Dimensions $13\frac{1}{2}'' \times 12''$ $\times$ 88 lbs. per foot.
Actual Dimensions $13\frac{3}{8}'' \times 11\frac{13}{16}'' \times$ 88 lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=25 feet to the ton, approx.)	88 lbs. per foot	131·4 kilos. per m.
Sectional Area	25·9 sq. ins.	167·4 sq. cm.
Greatest Moment of Inertia (Axis XX)	847 inches^4	35241 cm^4
Least ,, ,, (Axis YY)	195 ,,	8097 ,,
Greatest Section Modulus (Axis XX)	127 inches^3	2073 cm^3
Least ,, ,, (Axis YY)	33·0 ,,	540 ,,
Moment of Resistance (XX) at $7\frac{1}{2}$ tons stress	949 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	63 tons (10 ft.)	
Greatest Radius of Gyration (Axis XX)	5·71 inches	
Least ,, ,, (Axis YY)	2·74 ,,	
Maximum length rolled	about 68 feet	about 21 metres

The "Moment of Resistance" is the "Greatest Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: $7\frac{1}{2}$ tons per square inch).

Span (feet)	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons)	...	63	53	45	40	35	32	26	23	20	18	16	14	...
Deflection (inches)	...	·13	·19	·26	·34	·43	·53	·76	1·0	1·4	1·7	2·1	2·6	...

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about 18 inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·49 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet)	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons)	132	131	129	125	123	121	119	116	112	109	106	103	100	92
Nett Weight (cwts.)	6·30	7·09	7·88	8·66	9·45	10·2	11·0	11·8	12·6	13·4	14·2	15·0	15·8	17·3

Weight of Stanchion Base, about 3·37 cwts.; Cap, about $1\frac{1}{4}$ cwts.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·37 tons if applied to one of the flanges, or 1·21 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 148. Code Word: "ADMIRER."
Nominal Dimensions $13\frac{1}{2}'' \times 12'' \times 88$ lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

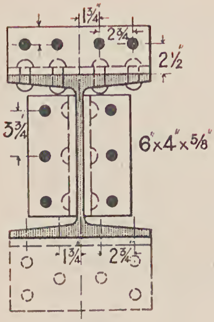


Fig. 1

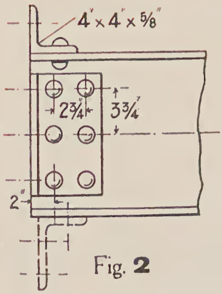


Fig. 2

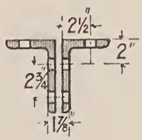


Fig. 3

SCALE-1/16

($\frac{3}{4}$ inch = 1 foot)

Rivets and Bolts $\frac{7}{8}''$

Bolt-Holes $\frac{15}{16}''$

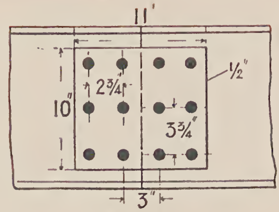


Fig 4

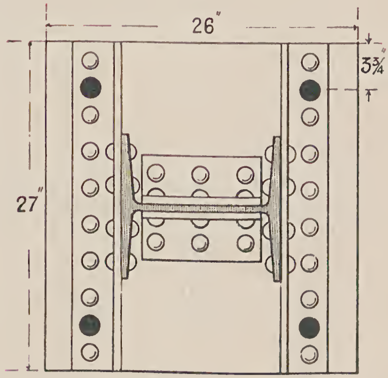


Fig. 8

Fig. 5

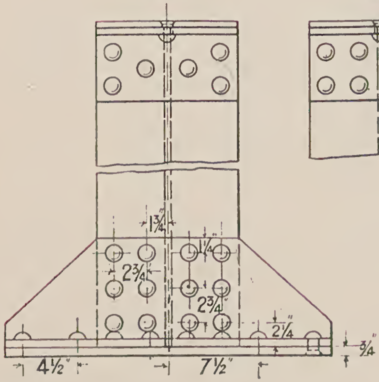


Fig. 6

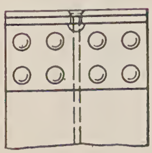
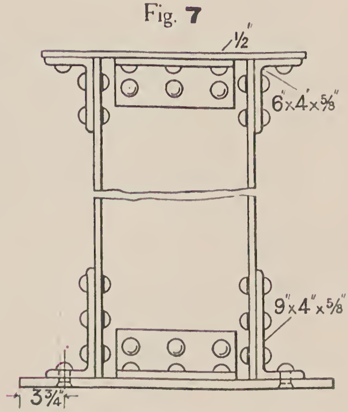


Fig. 7



Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 50·5 tons (span: $12\frac{1}{2}$ feet). The "Web Cleats" by themselves are suited to a distributed load of 28·9 tons only (span: 21·9 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, 4·88 square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 14·4 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 19·2 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 148. Code Word: "ADMIRER." Nominal Dimensions $13\frac{1}{2}'' \times 12'' \times 88$ lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.	LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 68.	Broad Flange Beam(s) $13\frac{1}{2}'' \times 12''$ approx. H. J. SKELTON & Co.'s Section No. 148	88 lbs. per ft.
	N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	25 ft. per ton
Figs. 1-3. Web Cleats	2 Angles $6'' \times 4'' \times \frac{5}{8}''$ by $10''$ long - -	33 lbs. per pair
	6 $\frac{7}{8}''$ Rivets, $1\frac{7}{8}''$ grip - - - - -	5 lbs.
	6 $\frac{7}{8}''$ Bolts - - - - - - -	9 lbs. say
	Gross weight of Web Cleats	47 lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1 Angle $4'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long - -	$15\frac{1}{2}$ lbs.
	4 $\frac{7}{8}''$ Rivets, $1\frac{5}{8}''$ grip - - - - -	3 lbs.
	4 $\frac{7}{8}''$ Bolts - - - - - - -	6 lbs. say
	Gross weight of one Upper Flange Cleat	$24\frac{1}{2}$ lbs.
Figs. 1, 2. Lower Flange Cleat	1 Angle $6'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long - -	$19\frac{1}{2}$ lbs.
	Gross weight including 4 Bolts & 6 Rivets	30 lbs. say
Fig. 4. Fishplates	2 $11'' \times 10'' \times \frac{1}{2}''$ - - - - - - -	31 lbs. per pair
	12 $\frac{7}{8}''$ Bolts, $2\frac{1}{2}''$ long - - - - -	14 lbs.
	Gross weight of Fishplates	45 lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2 Angles to flanges $6'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long	39 lbs. per pair
	2 Angles to web $4'' \times 4'' \times \frac{5}{8}''$ by $10''$ long -	26 lbs. per pair
	1 Cover Plate (if required) $22'' \times 12'' \times \frac{1}{2}''$ -	37 lbs.
	17 $\frac{7}{8}''$ Rivets (2 countersunk) - - -	13 lbs.
	(14) $\frac{7}{8}''$ Bolts - - - - - - -	22 lbs. say
	Gross weight of one Light Cap (including plate)	137 lbs.
Fig. 6. Stanchion Cap (heavy)	Same as for Light Cap, but 4 extra Rivets - - - - - -	3 lbs. extra
	Gross weight of one Heavy Cap (including plate)	140 lbs.
Figs. 5-8. Stanchion Base	1 Sole Plate $26'' \times 27'' \times \frac{3}{4}''$ - - - - -	149 lbs.
	2 Angles to flanges $9'' \times 4'' \times \frac{5}{8}''$ by $27''$ long	119 lbs. per pair
	2 Angles to web $4'' \times 4'' \times \frac{5}{8}''$ by $10''$ long -	26 lbs. per pair
	49 $\frac{7}{8}''$ Rivets (22 countersunk) - - -	34 lbs.
	4 Holding-down Bolts $1\frac{1}{4}'' \times 21''$ with $5'' \times 5'' \times \frac{3}{8}''$ plates - - - -	49 lbs.
	Gross weight of one Stanchion Base	377 lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 4 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

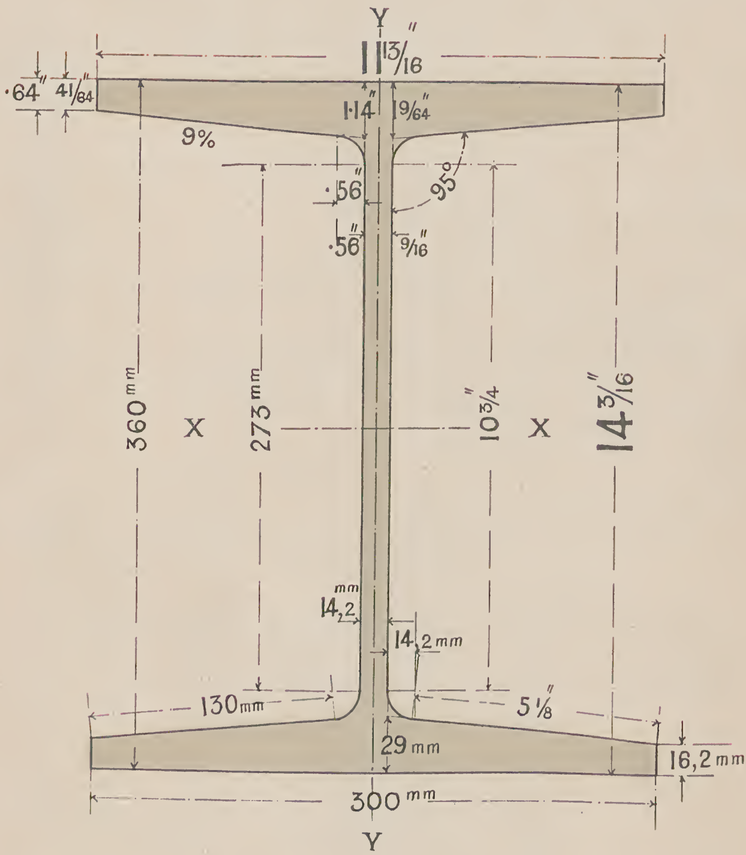
For further notes, see page 19.

SECTION DRAWING OF

B.F. BEAM { Section No. 152. Code Word: "ADVENT." Nominal Dimensions 14″×12″×96 lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]

STOCK SIZE.



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM { Section No. 152. Code Word: "ADVENT."
Nominal Dimensions 14" × 12" × 96 lbs. per foot.
Actual Dimensions $14\frac{3}{16}" \times 11\frac{13}{16}" \times 96$ lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=23 feet to the ton, approx.)	96 lbs. per foot	142·5 kilos. per m.
Sectional Area	28·1 sq. ins.	181·5 sq. cm.
Greatest Moment of Inertia (Axis XX)	1021 inches^4	42479 cm^4
Least " " (Axis YY)	211 "	8793 "
Greatest Section Modulus (Axis XX)	144 inches^3	2360 cm^3
Least " " (Axis YY)	35·8 "	586 "
Moment of Resistance (XX) at $7\frac{1}{2}$ tons stress	1080 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	71 tons (10 ft.)	
Greatest Radius of Gyration (Axis XX)	6·02 inches	
Least " " (Axis YY)	2·74 "	
Maximum length rolled	about 68 feet	about 21 metres

The "MOMENT OF RESISTANCE" is the "Greatest Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: $7\frac{1}{2}$ tons per square inch).

Span (feet)	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons)	...	*71*	60	51	45	40	36	30	26	23	20	18	16	...
Deflection (inches)	...	...	·18	·24	·32	·40	·50	·72	·98	1·3	1·6	2·0	2·4	...

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about $22\frac{1}{2}$ inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·49 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet)	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons)	143	142	139	136	133	130	129	125	121	118	115	112	108	100
Nett Weight (cwts.)	6·83	7·69	8·54	9·40	10·3	11·1	12·0	12·8	13·7	14·5	15·4	16·2	17·1	18·8

Weight of Stanchion Base, about 3·43 cwts.; Cap, about 1·28 cwts.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·38 tons if applied to one of the flanges, or 1·22 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

14" 15" 16" 17" 18" 19" 20" 22" 24" 26" 30"

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 152. Code Word: "ADVENT."
Nominal Dimensions 14″ × 12″ × 96 lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

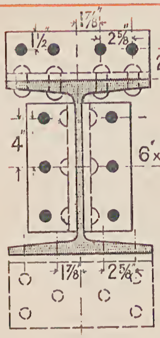


Fig. 1

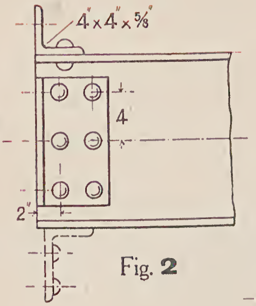


Fig. 2

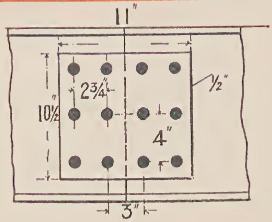


Fig 4

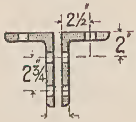


Fig. 3

SCALE 1/16

(¾ inch = 1 foot)

Rivets and Bolts ⅞″

Bolt-Holes $\frac{15}{16}$″

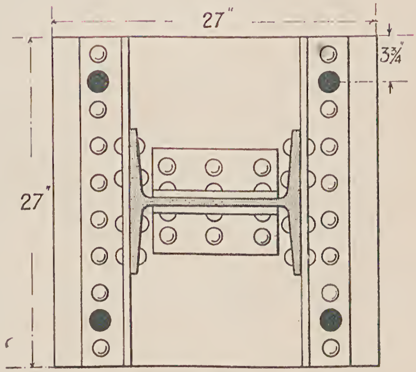


Fig. 8

Fig. 5

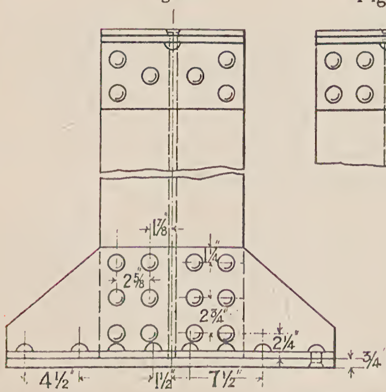


Fig. 6

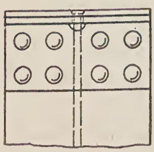
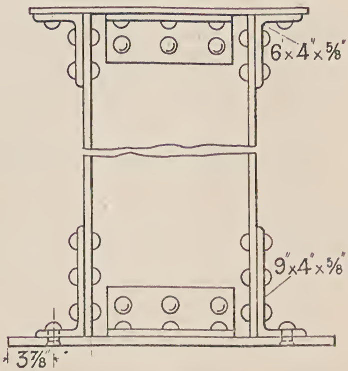


Fig. 7



Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 50·5 tons (span: 14·3 feet). The "Web Cleats" by themselves are suited to a distributed load of 28·9 tons only (span: 25 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, 5·06 square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 14·4 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 19·2 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 152. Code Word: "ADVENT."
Nominal Dimensions $14'' \times 12'' \times 96$ lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.	LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 72.	Broad Flange Beam(s) $14'' \times 12''$ approx. - H. J. SKELTON & Co.'s Section No. 152	96 lbs. per ft.
	N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	23 ft. per ton
Figs. 1-3. Web Cleats	2 Angles $6'' \times 4'' \times \frac{5}{8}''$ by $10\frac{1}{2}''$ long - -	35 lbs. per pair
	6 $\frac{7}{8}''$ Rivets, $1\frac{7}{8}''$ grip - - - - -	5 lbs.
	6 $\frac{7}{8}''$ Bolts - - - - - - -	9 lbs. say
	Gross weight of Web Cleats	49 lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1 Angle $4'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long - -	$15\frac{1}{2}$ lbs.
	4 $\frac{7}{8}''$ Rivets, $1\frac{5}{8}''$ grip - - - - -	3 lbs.
	4 $\frac{7}{8}''$ Bolts - - - - - - -	6 lbs. say
	Gross weight of one Upper Flange Cleat	$24\frac{1}{2}$ lbs.
Figs. 1, 2. Lower Flange Cleat	1 Angle $6'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long - -	$19\frac{1}{2}$ lbs.
	Gross weight including 4 Bolts & 6 Rivets	30 lbs. say
Fig. 4. Fishplates	2 $11'' \times 10\frac{1}{2}'' \times \frac{1}{2}''$ - - - - - - -	33 lbs. per pair
	12 $\frac{7}{8}''$ Bolts, $2\frac{1}{2}''$ long - - - - -	14 lbs.
	Gross weight of Fishplates	47 lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2 Angles to flanges $6'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long	39 lbs. per pair
	2 Angles to web $4'' \times 4'' \times \frac{5}{8}''$ by $10\frac{1}{2}''$ long -	$27\frac{1}{2}$ lbs. per pair
	1 Cover Plate (if required) $23'' \times 12'' \times \frac{1}{2}''$ -	39 lbs.
	17 $\frac{7}{8}''$ Rivets (2 countersunk) - - -	13 lbs.
	(14) $\frac{7}{8}''$ Bolts - - - - - - -	22 lbs. say
	Gross weight of one Light Cap (including plate)	$140\frac{1}{2}$ lbs.
Fig. 6. Stanchion Cap (heavy)	Same as for Light Cap, but 4 extra Rivets - - - - - -	3 lbs. extra
	Gross weight of one Heavy Cap (including plate)	$143\frac{1}{2}$ lbs.
Figs. 5-8. Stanchion Base	1 Sole Plate $27'' \times 27'' \times \frac{3}{4}''$ - - - - -	155 lbs.
	2 Angles to flanges $9'' \times 4'' \times \frac{5}{8}''$ by $27''$ long	119 lbs. per pair
	2 Angles to web $4'' \times 4'' \times \frac{5}{8}''$ by $10\frac{1}{2}''$ long -	$27\frac{1}{2}$ lbs. per pair
	49 $\frac{7}{8}''$ Rivets (22 countersunk) - - -	34 lbs.
	4 Holding-down Bolts $1\frac{1}{4}'' \times 21''$ with $5'' \times 5'' \times \frac{3}{8}''$ plates - - - -	49 lbs.
	Gross weight of one Stanchion Base	$384\frac{1}{2}$ lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 4 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

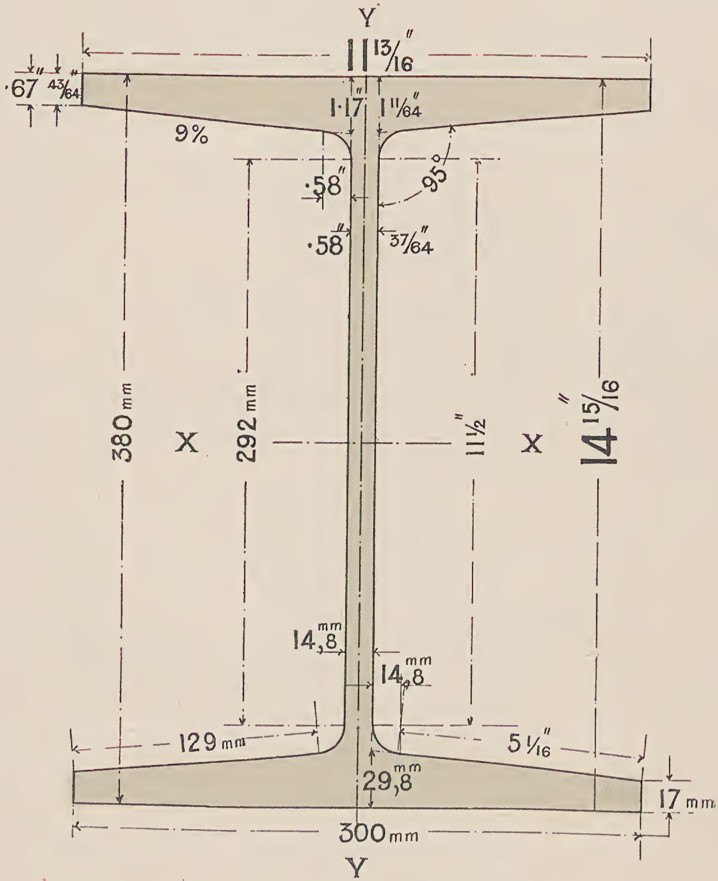
The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

For further notes, see page 19.

SECTION DRAWING OF

B.F. BEAM { Section No. 156. Code Word: "AGGRESSOR."
Nominal Dimensions 15″×12″×101 lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM
- Section No. 156. Code Word: "AGGRESSOR."
- Nominal Dimensions 15″ × 12″ × 101 lbs. per foot.
- Actual Dimensions $14\frac{15}{16}$″ × $11\frac{13}{16}$″ × 101 lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=22 feet to the ton, approx.) - -	101 lbs. per foot	150·1 kilos. per m.
Sectional Area - - - - - -	29·6 sq. ins.	191·2 sq. cm.
Greatest Moment of Inertia (Axis XX) - -	1190 inches^4	49496 cm^4
Least ,, ,, (Axis YY) - -	221 ,,	9175 ,,
Greatest Section Modulus (Axis XX) - -	159 inches^3	2605 cm^3
Least ,, ,, (Axis YY) - -	37·4 ,,	612 ,,
Moment of Resistance (XX) at 7½ tons stress -	1192 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	74 tons (11 ft.)	
Greatest Radius of Gyration (Axis XX) - -	6·33 inches	
Least ,, ,, (Axis YY) - -	2·73 ,,	
Maximum length rolled - - - - -	about 68 feet	about 21 metres

The "MOMENT OF RESISTANCE" is the "Greatest Section Modulus" multiplied by the working stress (here taken as 7½ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: 7½ tons per square inch).

Span (feet) - -	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons) -	...	*74*	66	57	50	44	40	33	28	25	22	20	18	...
Deflection (inches)	...	...	·17	·23	·30	·38	·47	·68	·93	1·2	1·5	1·9	2·3	...

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about 22½ inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·49 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet) -	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons) -	151	150	146	143	140	138	135	131	128	124	121	118	114	104
Nett Weight (cwts.)	7·20	8·10	9·00	9·90	10·8	11·7	12·6	13·5	14·4	15·3	16·2	17·1	18·0	19·8

Weight of Stanchion Base, about 3·89 cwts.; Cap, about 1·37 cwts.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·39 tons if applied to one of the flanges, or 1·23 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 156. Code Word: "AGGRESSOR."
Nominal Dimensions 15″×12″×101 lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

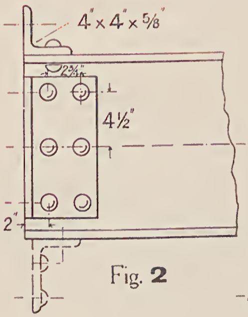


Fig. 2

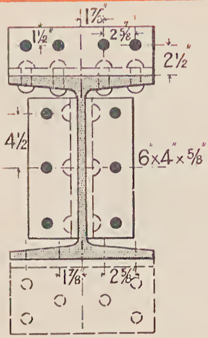


Fig. 1

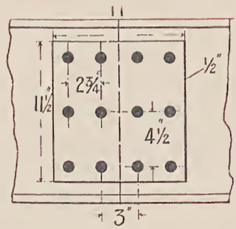


Fig 4

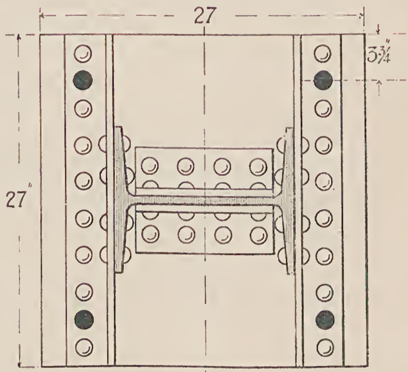


Fig. 8

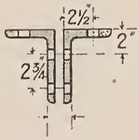


Fig. 3

SCALE 1/16

($\frac{3}{4}$ inch = 1 foot)

Rivets and Bolts $\frac{7}{8}$″

Bolt-Holes $\frac{15}{16}$″

Fig. 5

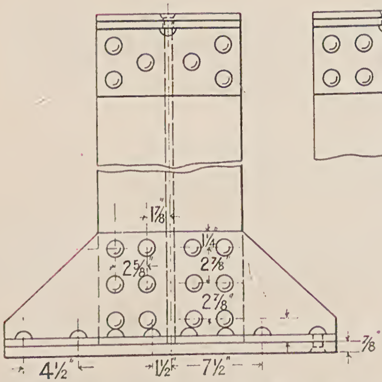


Fig. 6

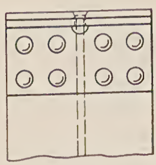
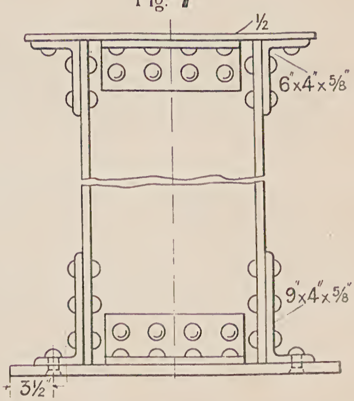


Fig. 7



Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 50·5 tons (span: 15·7 feet). The "Web Cleats" by themselves are suited to a distributed load of 28·9 tons only (span: 27½ feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, 5·06 square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 14·4 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 19·2 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 156. Code Word: "AGGRESSOR." Nominal Dimensions 15″×12″×101 lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.		LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 76.		Broad Flange Beam(s) 15″ × 12″ approx. - H. J. SKELTON & Co.'s Section No. 156	101 lbs. per ft.
		N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	22 ft. per ton
Figs. 1-3. Web Cleats	2	Angles 6″ × 4″ × ⅝″ by 11½″ long - -	38 lbs. per pair
	6	⅞″ Rivets, 1⅞″ grip - - - - -	5 lbs.
	6	⅞″ Bolts - - - - - - -	9 lbs. say
		Gross weight of Web Cleats	52 lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1	Angle 4″ × 4″ × ⅝″ by 11¾″ long - -	15½ lbs.
	4	⅞″ Rivets, 1¾″ grip - - - - -	3 lbs.
	4	⅞″ Bolts - - - - - - -	6 lbs. say
		Gross weight of one Upper Flange Cleat	24½ lbs.
Figs. 1, 2. Lower Flange Cleat	1	Angle 6″ × 4″ × ⅝″ by 11¾″ long - -	19½ lbs.
		Gross weight including 4 Bolts & 6 Rivets	30 lbs. say
Fig. 4. Fishplates	2	11″ × 11½″ × ½″ - - - - -	36 lbs. per pair
	12	⅞″ Bolts, 2½″ long - - - - -	14 lbs.
		Gross weight of Fishplates	50 lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2	Angles to flanges 6″ × 4″ × ⅝″ by 11¾″ long	39 lbs. per pair
	2	Angles to web 4″ × 4″ × ⅝″ by 11½″ long -	30 lbs. per pair
	1	Cover Plate (if required) 24″ × 12″ × ½″ -	41 lbs.
	18	⅞″ Rivets (2 countersunk) - - -	14 lbs.
	(16)	⅞″ Bolts - - - - - - -	26 lbs. say
		Gross weight of one Light Cap (including plate)	150 lbs.
Fig. 6. Stanchion Cap (heavy)		Same as for Light Cap, but 4 extra Rivets - - - - - -	3 lbs. extra
		Gross weight of one Heavy Cap (including plate)	153 lbs.
Figs. 5-8. Stanchion Base	1	Sole Plate 27″ × 27″ × ⅞″ - - - -	181 lbs.
	2	Angles to flanges 9″ × 4″ × ⅝″ by 27″ long	119 lbs. per pair
	2	Angles to web 4″ × 4″ × ⅝″ by 11½″ long -	30 lbs. per pair
	52	⅞″ Rivets (24 countersunk) - - -	38 lbs.
	4	Holding - down Bolts 1½″ × 21″ with 5″ × 5″ × ⅜″ plates - - - -	68 lbs.
		Gross weight of one Stanchion Base	436 lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 4 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

For further notes, see page 19.

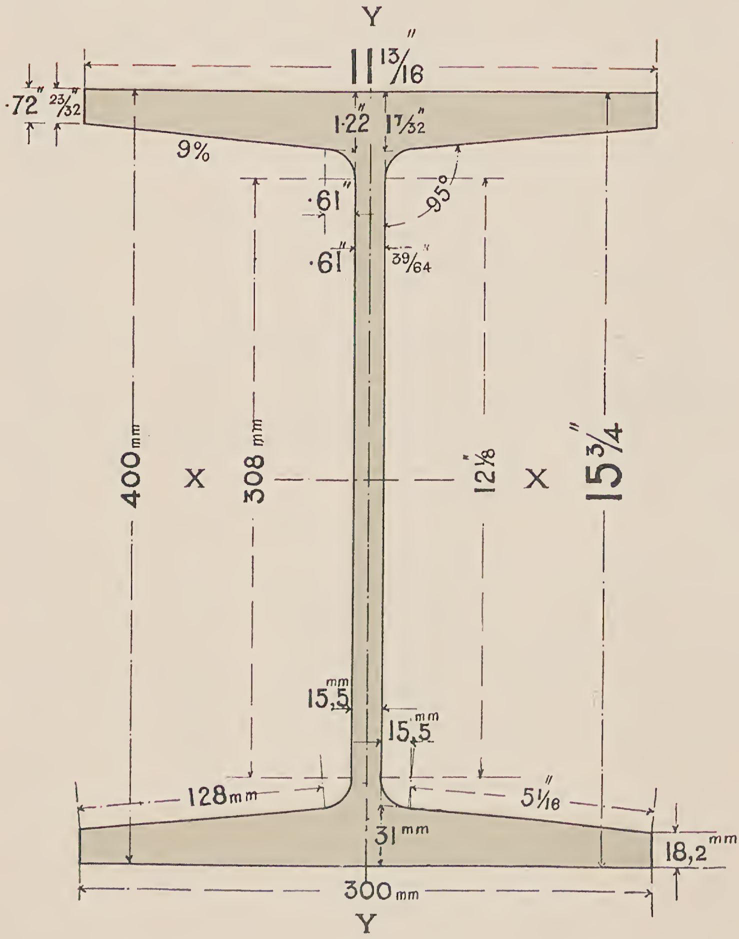
16″ 17″ 18″ 19″ 20″ 22″ 24″ 26″ 30″

SECTION DRAWING OF

B.F. BEAM { Section No. 160. Code Word: "AGNOSTIC."
Nominal Dimensions 16″×12″×107 lbs. per foot.

[For Properties, see opposite page.] For Drawings of Connections, P.T.O.]

STOCK SIZE.



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM { Section No. 160. Code Word: "AGNOSTIC."
Nominal Dimensions 16″ ×12″ ×107 lbs. per foot.
Actual Dimensions $15\frac{3}{4}''\times11\frac{13}{16}''\times107$ lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=21 feet to the ton, approx.) - -	107 lbs. per foot	159·8 kilos. per m.
Sectional Area - - - - - -	31·6 sq. ins.	203·6 sq. cm.
Greatest Moment of Inertia (Axis XX) - -	1390 inches^4	57834 cm^4
Least ,, ,, (Axis YY) - -	234 ,,	9721 ,,
Greatest Section Modulus (Axis XX) - -	177 inches^3	2892 cm^3
Least ,, ,, (Axis YY) - -	39·6 ,,	648 ,,
Moment of Resistance (XX) at $7\frac{1}{2}$ tons stress -	1324 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	82 tons (11 ft.)	
Greatest Radius of Gyration (Axis XX) - -	6·63 inches	
Least ,, ,, (Axis YY) - -	2·72 ,,	
Maximum length rolled - - - - -	about 68 feet	about 21 metres

The "MOMENT OF RESISTANCE" is the "Greatest Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: $7\frac{1}{2}$ tons per square inch).

Span (feet) - -	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons) -	...	*82*	73	63	55	49	44	37	31	28	25	22	20	...
Deflection (inches)	...	...	·16	·22	·29	·36	·45	·65	·88	1·2	1·4	1·8	2·2	...

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about $22\frac{1}{2}$ inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about $\frac{1}{2}$ cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet) -	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons) -	161	159	155	152	149	147	145	140	136	132	129	125	121	111
Nett Weight (cwts.)	7·66	8·62	9·58	10·5	11·5	12·5	13·4	14·4	15·3	16·3	17·3	18·2	19·2	21·1

Weight of Stanchion Base, about 4·74 cwts.; Cap, about 1·55 cwts.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·41 tons if applied to one of the flanges, or 1·24 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 160. Code Word: "AGNOSTIC."
Nominal Dimensions 16"×12"×107 lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

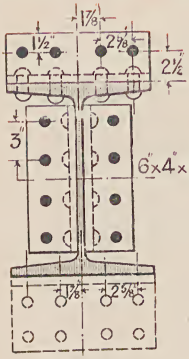


Fig. 1

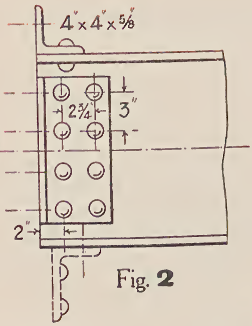


Fig. 2

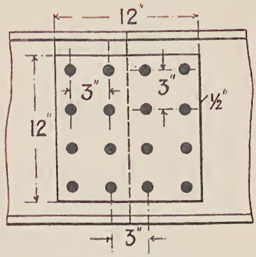


Fig 4

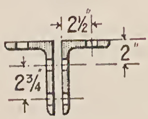


Fig. 3

SCALE 1/16

($\frac{3}{4}$ inch = 1 foot)

Rivets and Bolts $\frac{7}{8}$"

Bolt-Holes $\frac{15}{16}$"

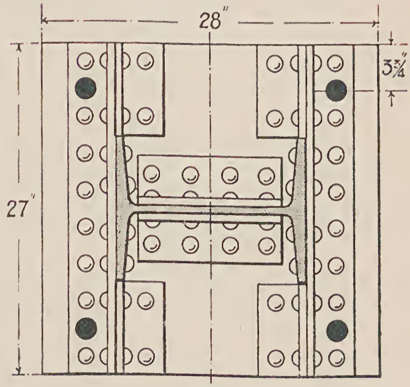


Fig. 8

Fig. 5

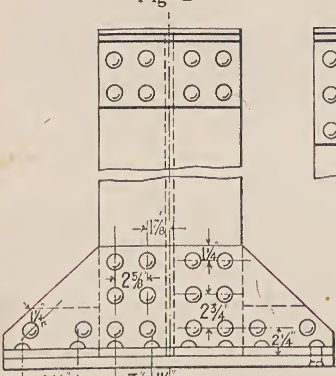


Fig. 6

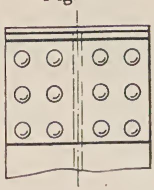
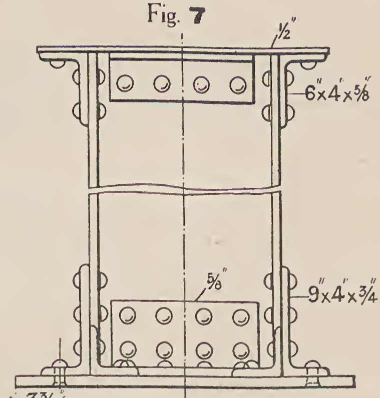


Fig. 7



Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 67·3 tons (span: 13·1 feet). The "Web Cleats" by themselves are suited to a distributed load of 38·5 tons only (span: 22·9 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, $5\frac{1}{4}$ square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 19·2 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 28·9 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 160. Code Word: "AGNOSTIC."
Nominal Dimensions $16'' \times 12'' \times 107$ lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.		LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 80.		Broad Flange Beam(s) $16'' \times 12''$ approx. - H. J. SKELTON & Co.'s Section No. 160	107 lbs. per ft.
		N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	21 ft. per ton
Figs. 1-3. Web Cleats	2	Angles $6'' \times 4'' \times \frac{5}{8}''$ by $12''$ long - -	40 lbs. per pair
	8	$\frac{7}{8}''$ Rivets, $1\frac{7}{8}''$ grip - - - - -	$6\frac{1}{2}$ lbs.
	8	$\frac{7}{8}''$ Bolts - - - - - - -	12 lbs. say
		Gross weight of Web Cleats	$58\frac{1}{2}$ lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1	Angle $4'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long - -	$15\frac{1}{2}$ lbs.
	4	$\frac{7}{8}''$ Rivets, $1\frac{3}{4}''$ grip - - - - -	3 lbs.
	4	$\frac{7}{8}''$ Bolts - - - - - - -	6 lbs. say
		Gross weight of one Upper Flange Cleat	$24\frac{1}{2}$ lbs.
Figs. 1, 2. Lower Flange Cleat	1	Angle $6'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long - -	$19\frac{1}{2}$ lbs.
		Gross weight including 4 Bolts & 8 Rivets	32 lbs. say
Fig. 4. Fishplates	2	$12'' \times 12'' \times \frac{1}{2}''$ - - - - - - -	41 lbs. per pair
	16	$\frac{7}{8}''$ Bolts, $3''$ long - - - - - -	20 lbs.
		Gross weight of Fishplates	61 lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2	Angles to flanges $6'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long	39 lbs. per pair
	2	Angles to web $4'' \times 4'' \times \frac{5}{8}''$ by $12''$ long -	$31\frac{1}{2}$ lbs. per pair
	1	Cover Plate (if required) $24'' \times 12'' \times \frac{1}{2}''$ -	41 lbs.
	22	$\frac{7}{8}''$ Rivets (2 countersunk) - -	17 lbs.
	(16)	$\frac{7}{8}''$ Bolts - - - - - - -	26 lbs. say
		Gross weight of one Light Cap (including plate)	$154\frac{1}{2}$ lbs.
Fig. 6. Stanchion Cap (heavy)	2	Angles to flanges $9'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long	52 lbs. per pair
		Angles to web, Bolts and Plate as for Light Cap - - - - - -	$98\frac{1}{2}$ lbs.
	30	$\frac{7}{8}''$ Rivets (2 countersunk) - - -	23 lbs.
		Gross weight of one Heavy Cap (including plate)	$173\frac{1}{2}$ lbs.
Figs. 5-8. Stanchion Base	1	Sole Plate $28'' \times 27'' \times \frac{7}{8}''$ - - - -	187 lbs.
	2	Angles to flanges $9'' \times 4'' \times \frac{3}{4}''$ by $27''$ long -	141 lbs. per pair
	2	Angles to web $6'' \times 4'' \times \frac{5}{8}''$ by $12''$ long -	40 lbs. per pair
	4	Angle Spreaders $4'' \times 4'' \times \frac{5}{8}''$ by $7\frac{1}{2}''$ long -	39 lbs. total
	72	$\frac{7}{8}''$ Rivets (32 countersunk) - - -	53 lbs.
	4	Holding-down Bolts $1\frac{1}{2}'' \times 21''$ with $5'' \times 5'' \times \frac{1}{2}''$ plates - - - -	71 lbs.
		Gross weight of one Stanchion Base	531 lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 4 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

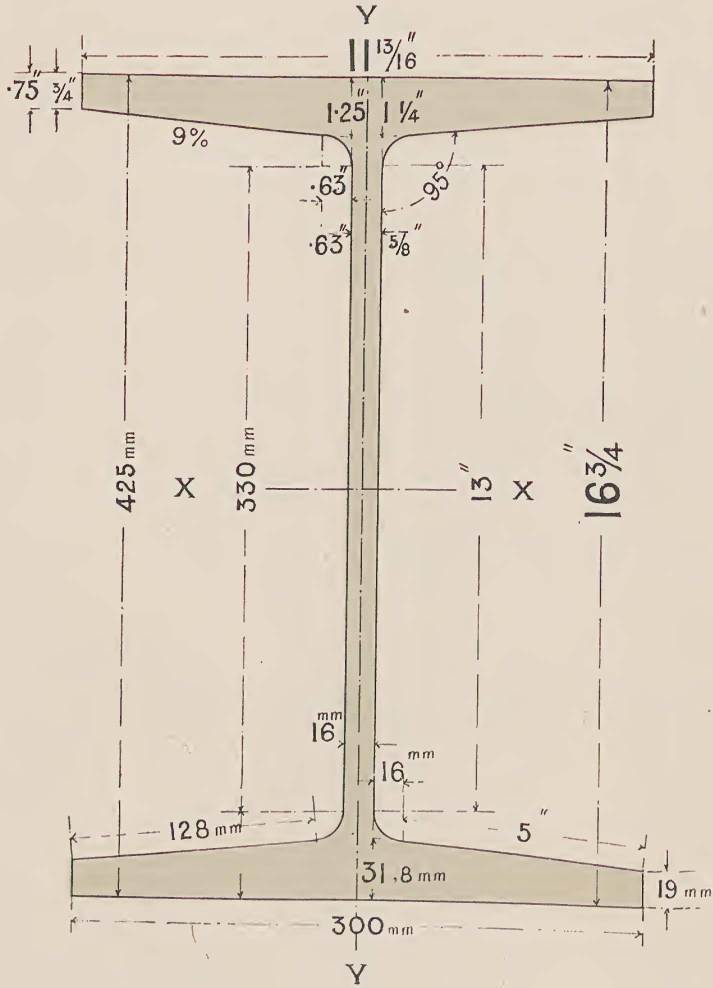
The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

For further notes, see page 19.

SECTION DRAWING OF

B.F. BEAM { Section No. 164. Code Word: "ALABASTER."
Nominal Dimensions 17″ × 12″ × 113 lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM
- Section No. 164. Code Word: "ALABASTER."
- Nominal Dimensions 17" × 12" × 113 lbs. per foot.
- Actual Dimensions $16\frac{3}{4}$" × $11\frac{13}{16}$" × 113 lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=20 feet to the ton, approx.) - -	113 lbs. per foot	167·9 kilos. per m.
Sectional Area - - - - - -	33·2 sq. ins.	213·9 sq. cm.
Greatest Moment of Inertia (Axis XX) - -	1640 inches^4	68249 cm^4
Least " " (Axis YY) - -	242 "	10078 "
Greatest Section Modulus (Axis XX) - -	196 inches^3	3212 cm^3
Least " " (Axis YY) - -	41·0 "	672 "
Moment of Resistance (XX) at $7\frac{1}{2}$ tons stress -	1470 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	87 tons (11 ft.)	
Greatest Radius of Gyration (Axis XX) - -	7·03 inches	
Least " " (Axis YY) - -	2·70 "	
Maximum length rolled - - - - -	about 68 feet	about 21 metres

The "MOMENT OF RESISTANCE" is the "Greatest Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: $7\frac{1}{2}$ tons per square inch).

Span (feet) - -	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons) -	...	*87*	82	70	61	54	49	41	35	31	27	24	22	19
Deflection (inches)	...	...	·15	·21	·27	·34	·42	·61	·83	1·1	1·4	1·7	2·0	2·9

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about $22\frac{1}{2}$ inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about $\frac{1}{2}$ cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet) -	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons) -	169	167	163	160	157	154	151	147	143	139	135	131	126	116
Nett Weight (cwts.)	8·06	9·06	10·1	11·1	12·1	13·1	14·1	15·1	16·1	17·1	18·1	19·1	20·1	22·2

Weight of Stanchion Base, about 5·73 cwts.; Cap, about 1·59 cwts.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·42 tons if applied to one of the flanges, or 1·26 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 164. Code Word: "ALABASTER."
Nominal Dimensions 17″ × 12″ × 113 lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

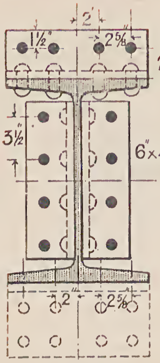


Fig. **1**

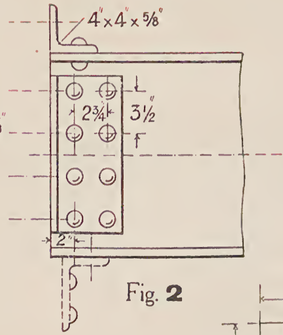


Fig. **2**

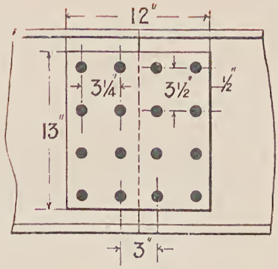


Fig. **4**

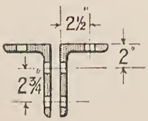


Fig. **3**

SCALE 1/16

(¾ inch = 1 foot)

Rivets and Bolts ⅞″

Bolt-Holes $\frac{15}{16}$″

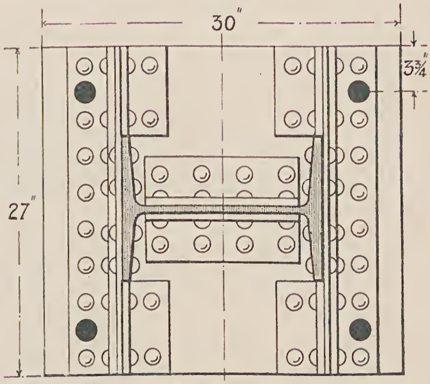


Fig. **8**

Fig. **5**

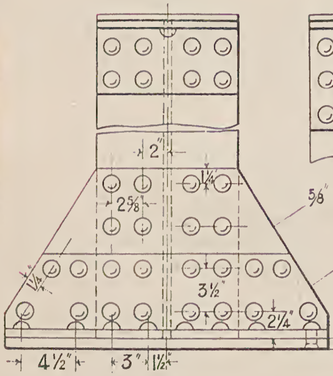


Fig. **6**

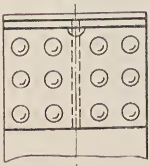
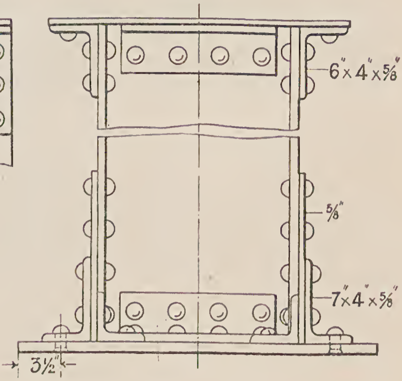


Fig. **7**



Figs. 1-3. End Connection for a Girder. The Combined "Web and Flange Cleats" are suited to a distributed load of about 67·3 tons (span: 14·6 feet). The "Web Cleats" by themselves are suited to a distributed load of 38·5 tons only (span: 25½ feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, 5·63 square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 19·2 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 28·9 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 164. Code Word: "ALABASTER." Nominal Dimensions 17″ × 12″ × 113 lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.		LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 84.		Broad Flange Beam(s) 17″ × 12″ approx. - H. J. SKELTON & Co.'s Section No. 164	113 lbs. per ft.
		N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	20 ft. per ton
Figs. 1-3. Web Cleats	2	Angles 6″ × 4″ × 5/8″ by 13″ long - -	43 lbs. per pair
	8	7/8″ Rivets, 1 7/8″ grip - - - - -	6½ lbs.
	8	7/8″ Bolts - - - - - - -	12 lbs. say
		Gross weight of Web Cleats	61½ lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1	Angle 4″ × 4″ × 5/8″ by 11¾″ long - -	15½ lbs.
	4	7/8″ Rivets, 1¾″ grip - - - - -	3 lbs.
	4	7/8″ Bolts - - - - - - -	6 lbs. say
		Gross weight of one Upper Flange Cleat	24½ lbs.
Figs. 1, 2. Lower Flange Cleat	1	Angle 6″ × 4″ × 5/8″ by 11¾″ long - -	19½ lbs.
		Gross weight including 4 Bolts & 8 Rivets	32 lbs. say
Fig. 4. Fishplates	2	12″ × 13″ × ½″ - - - - - - -	44 lbs. per pair
	16	7/8″ Bolts, 3″ long - - - - - -	20 lbs.
		Gross weight of Fishplates	64 lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2	Angles to flanges 6″ × 4″ × 5/8″ by 11¾″ long	39 lbs. per pair
	2	Angles to web 4″ × 4″ × 5/8″ by 13″ long -	34 lbs. per pair
	1	Cover Plate (if required) 25″ × 12″ × ½″ -	43 lbs.
	22	7/8″ Rivets (2 countersunk) - - -	17 lbs.
	(16)	7/8″ Bolts and Nuts - - - - -	26 lbs. say
		Gross weight of one Light Cap (including plate)	159 lbs.
Fig. 6. Stanchion Cap (heavy)	2	Angles to flanges 9″ × 4″ × 5/8″ by 11¾″ long	52 lbs. per pair
		Angles to web, Bolts and Plate as for Light Cap - - - - - -	103 lbs.
	30	7/8″ Rivets (2 countersunk) - - -	23 lbs.
		Gross weight of one Heavy Cap (including plate)	178 lbs.
Figs. 5-8. Stanchion Base	1	Sole Plate 30″ × 27″ × 7/8″ - - - -	201 lbs.
	2	Gusset Plates 27″ × 14″ × 5/8″ - - -	134 lbs. per pair
	2	Angles to flanges 7″ × 4″ × 5/8″ by 27″ long	100 lbs. per pair
	2	Angles to web 4″ × 4″ × 5/8″ by 13″ long -	34 lbs. per pair
	4	Angle Spreaders 4″ × 4″ × 5/8″ by 7½″ long -	39 lbs. total
	84	7/8″ Rivets (32 countersunk) - - -	63 lbs.
	4	Holding - down Bolts 1½″ × 21″ with 5″ × 5″ × ½″ plates - - - -	71 lbs.
		Gross weight of one Stanchion Base	642 lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 5 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

For further notes, see page 19.

18″

19″

20″

22″

24″

26″

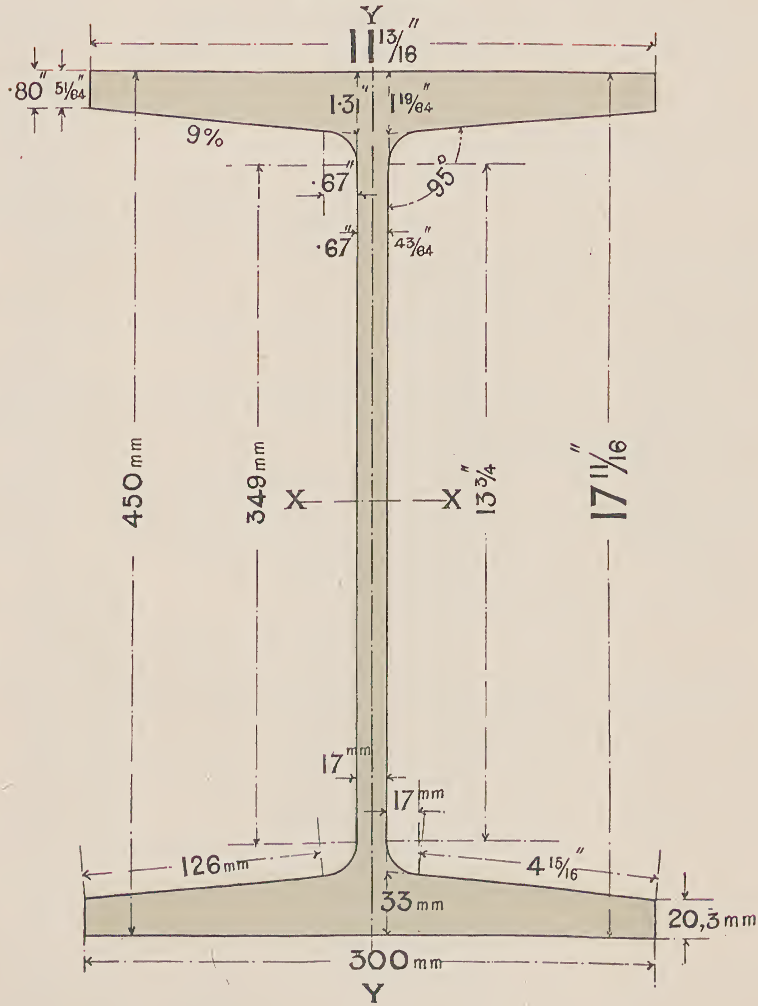
30″

SECTION DRAWING OF

B.F. BEAM { Section No. 168. Code Word: "ALARMIST."
Nominal Dimensions 18"×12"×121 lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]

STOCK SIZE.



SCALE 1/4 (3 inches = 1 foot)

PROPERTIES AND SAFE LOADS OF

B.F. BEAM { Section No. 168. Code Word : "ALARMIST."
Nominal Dimensions 18" × 12" × 121 lbs. per foot.
Actual Dimensions $17\frac{11}{16}$" × $11\frac{13}{16}$" × 121 lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=$18\frac{1}{2}$ feet to the ton, approx.)	121 lbs. per foot	180·0 kilos. per m.
Sectional Area	35·6 sq. ins.	229·3 sq. cm.
Greatest Moment of Inertia (Axis XX)	1944 inches^4	80887 cm^4
Least " " (Axis YY)	256 "	10668 "
Greatest Section Modulus (Axis XX)	219 inches^3	3595 cm^3
Least " " (Axis YY)	43·4 "	711 "
Moment of Resistance (XX) at $7\frac{1}{2}$ tons stress	1646 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	99 tons (11 ft.)	
Greatest Radius of Gyration (Axis XX)	7·39 inches	
Least " " (Axis YY)	2·68 "	
Maximum length rolled	about 68 feet	about 21 metres

The "MOMENT OF RESISTANCE" is the "Greatest Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress : $7\frac{1}{2}$ tons per square inch).

Span (feet)	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons)	...	*99*	92	78	69	61	55	46	39	34	30	27	25	21
Deflection (inches)	...	...	·14	·20	·25	·32	·40	·57	·78	1·0	1·3	1·6	1·9	2·7

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about $22\frac{1}{2}$ inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about $\frac{1}{2}$ cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet)	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons)	181	180	175	171	168	165	162	157	153	148	144	139	135	123
Nett Weight (cwts.)	8·63	9·71	10·8	11·9	13·0	14·0	15·1	16·2	17·3	18·3	19·4	20·5	21·6	23·7

Weight of Stanchion Base, about 5·74 cwts. ; Cap, about 1·62 cwts.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·44 tons if applied to one of the flanges, or 1·28 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

18" 19" 20" 22" 24" 26" 30"

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 168. Code Word: "ALARMIST."
Nominal Dimensions 18″ × 12″ × 121 lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

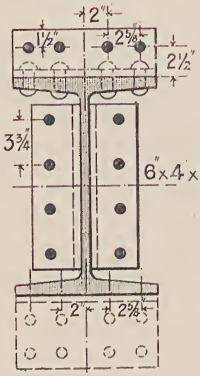


Fig. 1

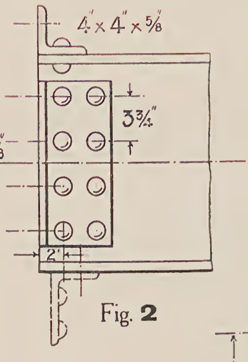


Fig. 2

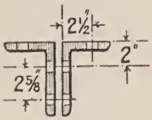


Fig. 3

SCALE 1/16

(¾ inch = 1 foot)

Rivets and Bolts ⅞″

Bolt-Holes $\frac{15}{16}$″

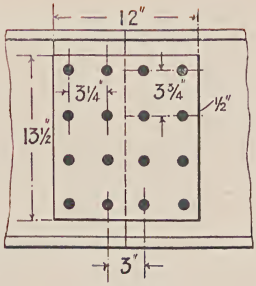


Fig. 4

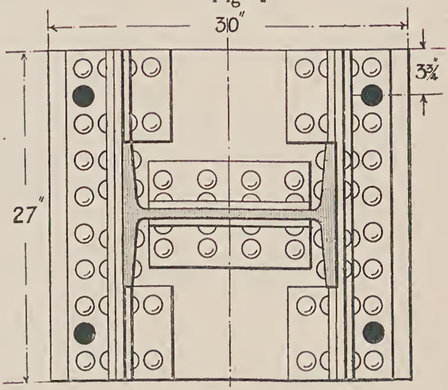


Fig. 8

Fig. 5

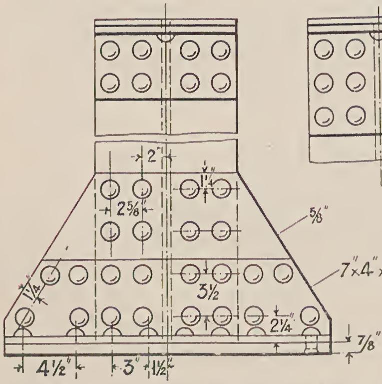


Fig. 6

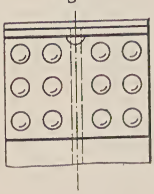
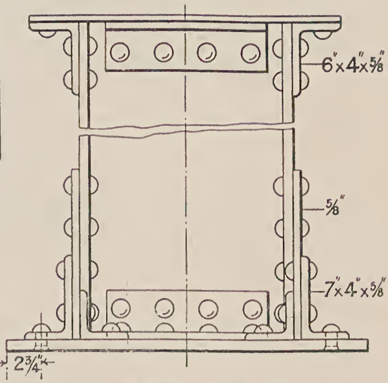


Fig. 7



Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 67·3 tons (span: 16·3 feet). The "Web Cleats" by themselves are suited to a distributed load of 38·5 tons only (span: 28·5 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, 5·63 square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 19·2 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 28·9 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 168. Code Word: "ALARMIST."
Nominal Dimensions $18'' \times 12'' \times 121$ lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.		LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 88.		Broad Flange Beam(s) $18'' \times 12''$ approx. - H. J. SKELTON & Co.'s Section No. 168	121 lbs. per ft.
		N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	$18\frac{1}{2}$ ft. per ton
Figs. 1-3. Web Cleats	2	Angles $6'' \times 4'' \times \frac{5}{8}''$ by $13\frac{1}{2}''$ long - -	45 lbs. per pair
	8	$\frac{7}{8}''$ Rivets, $2''$ grip - - - - -	$6\frac{1}{2}$ lbs.
	8	$\frac{7}{8}''$ Bolts - - - - - - -	12 lbs. say
		Gross weight of Web Cleats	$63\frac{1}{2}$ lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1	Angle $4'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long - -	$15\frac{1}{2}$ lbs.
	4	$\frac{7}{8}''$ Rivets, $1\frac{7}{8}''$ grip - - - - -	3 lbs.
	4	$\frac{7}{8}''$ Bolts - - - - - -	6 lbs. say
		Gross weight of one Upper Flange Cleat	$24\frac{1}{2}$ lbs.
Figs. 1, 2. Lower Flange Cleat	1	Angle $6'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long - -	$19\frac{1}{2}$ lbs.
		Gross weight including 4 Bolts & 8 Rivets	32 lbs. say
Fig. 4. Fishplates	2	$12'' \times 13\frac{1}{2}'' \times \frac{1}{2}''$ - - - - - -	46 lbs. per pair
	16	$\frac{7}{8}''$ Bolts, $3''$ long - - - - -	20 lbs.
		Gross weight of Fishplates	66 lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2	Angles to flanges $6'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long	39 lbs. per pair
	2	Angles to web $4'' \times 4'' \times \frac{5}{8}''$ by $13\frac{1}{2}''$ long -	35 lbs. per pair
	1	Cover Plate (if required) $26'' \times 12'' \times \frac{1}{2}''$ -	44 lbs.
	22	$\frac{7}{8}''$ Rivets (2 countersunk) - - -	$17\frac{1}{2}$ lbs.
	(16)	$\frac{7}{8}''$ Bolts - - - - - - -	26 lbs. say
		Gross weight of one Light Cap (including plate)	$161\frac{1}{2}$ lbs.
Fig. 6. Stanchion Cap (heavy)	2	Angles to flanges $9'' \times 4'' \times \frac{5}{8}''$ by $11\frac{3}{4}''$ long	52 lbs. per pair
		Angles to web, Bolts and Plate as for Light Cap - - - - - -	105 lbs.
	30	$\frac{7}{8}''$ Rivets (2 countersunk) - - -	24 lbs.
		Gross weight of one Heavy Cap (including plate)	181 lbs.
Figs. 5-8. Stanchion Base	1	Sole Plate $30'' \times 27'' \times \frac{7}{8}''$ - - - - -	201 lbs.
	2	Gusset Plates $27'' \times 14'' \times \frac{5}{8}''$ - - - -	134 lbs. per pair
	2	Angles to flanges $7'' \times 4'' \times \frac{5}{8}''$ by $27''$ long	100 lbs. per pair
	2	Angles to web $4'' \times 4'' \times \frac{5}{8}''$ by $13\frac{1}{2}''$ long -	35 lbs. per pair
	4	Angle Spreaders $4'' \times 4'' \times \frac{5}{8}''$ by $7\frac{1}{2}''$ long -	39 lbs. total
	84	$\frac{7}{8}''$ Rivets (32 countersunk) - - -	63 lbs.
	4	Holding-down Bolts $1\frac{1}{2}'' \times 21''$ with $5'' \times 5'' \times \frac{1}{2}''$ plates - - - -	71 lbs.
		Gross weight of one Stanchion Base	643 lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 5 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

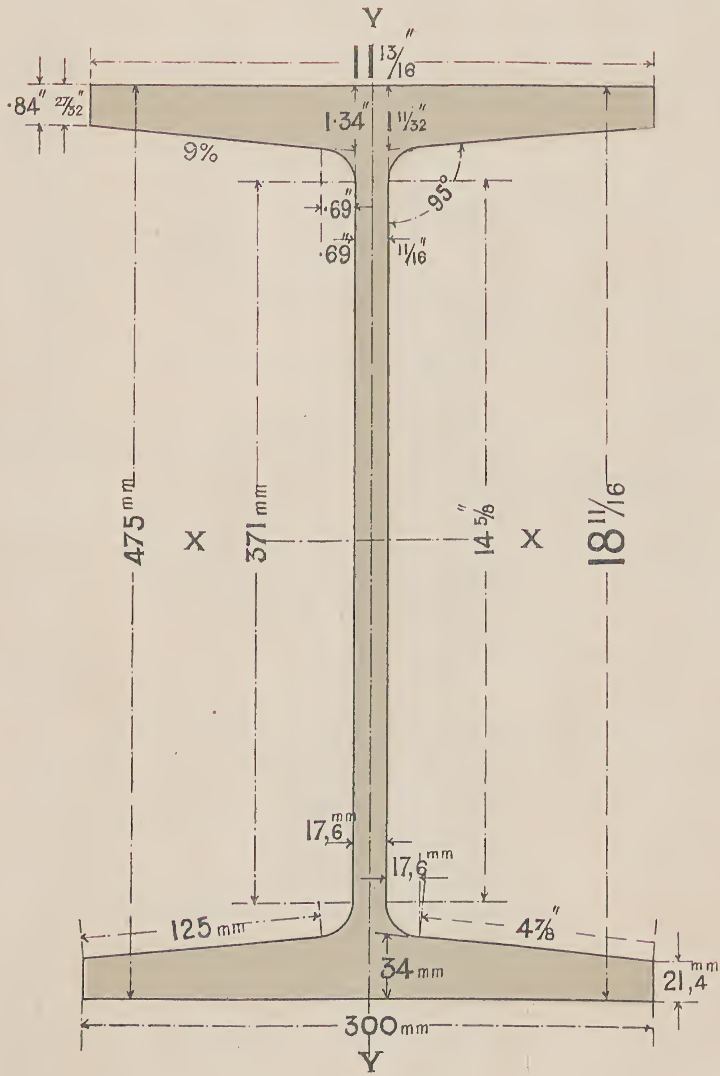
The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

For further notes, see page 19.

SECTION DRAWING OF

B.F. BEAM { Section No. 172. Code Word: "ALBATROSS." Nominal Dimensions 19″ × 12″ × 128 lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM
- Section No. 172. Code Word: "ALBATROSS."
- Nominal Dimensions 19" × 12" × 128 lbs. per foot.
- Actual Dimensions $18\frac{11}{16}$" × $11\frac{13}{16}$" × 128 lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=$17\frac{1}{2}$ feet to the ton, approx.) - -	128 lbs. per foot	190·0 kilos. per m.
Sectional Area - - - - - -	37·5 sq. ins.	242·0 sq. cm.
Greatest Moment of Inertia (Axis XX) - -	2278 inches^4	94811 cm^4
Least " " (Axis YY) - -	268 "	11142 "
Greatest Section Modulus (Axis XX) - -	244 inches^3	3992 cm^3
Least " " (Axis YY) - -	45·3 "	743 "
Moment of Resistance (xx) at $7\frac{1}{2}$ tons stress -	1827 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	103 tons (12 ft.)	
Greatest Radius of Gyration (Axis XX) - -	7·79 inches	
Least " " (Axis YY) - -	2·67 "	
Maximum length rolled - - - - -	about 68 feet	about 21 metres

The "MOMENT OF RESISTANCE" is the "Greatest Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: $7\frac{1}{2}$ tons per square inch).

Span (feet) - -	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons) -	...	*103*	101	87	76	68	61	51	43	38	34	30	28	23
Deflection (inches)	...	...	·14	·19	·24	·31	·38	·54	·74	·97	1·2	1·5	1·8	2·6

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about $22\frac{1}{2}$ inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about $\frac{1}{2}$ cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet) -	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons) -	191	189	184	181	177	174	170	165	161	156	152	147	142	129
Nett Weight (cwts.)	9·11	10·3	11·4	12·5	13·7	14·8	16·0	17·1	18·2	19·4	20·5	21·6	22·8	25·1

Weight of Stanchion Base, about 6·04 cwts.; Cap, about 1·66 cwts.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·44 tons if applied to one of the flanges, or 1·29 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

DRAWINGS OF CONNECTIONS SUITED TO

.. B.F. BEAM { Section No. 172. Code Word: "ALBATROSS." Nominal Dimensions 19″ × 12″ × 128 lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

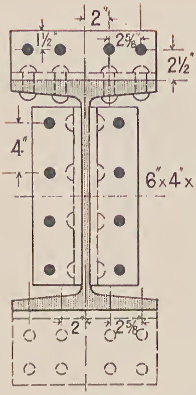


Fig. **1**

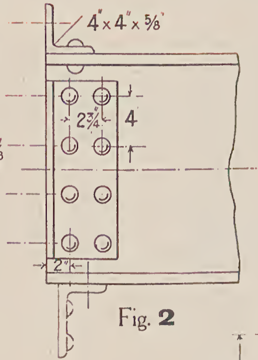


Fig. **2**

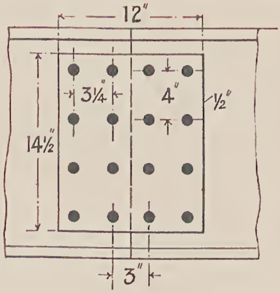


Fig. **4**

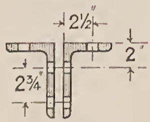


Fig. **3**

SCALE 1/16

(¾ inch = 1 foot)

Rivets and Bolts ⅞″

Bolt-Holes $\frac{15}{16}$″

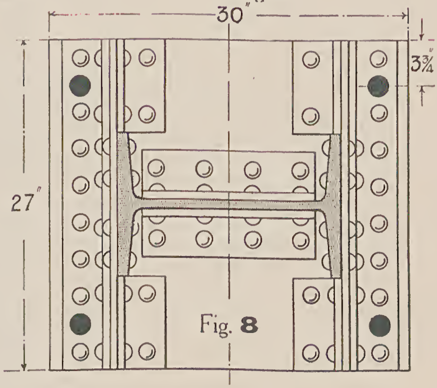


Fig. **8**

Fig. **5**

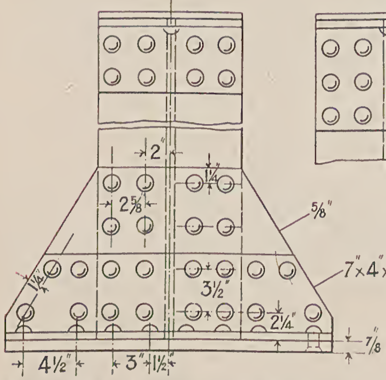


Fig. **6**

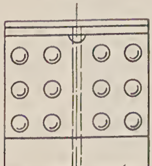
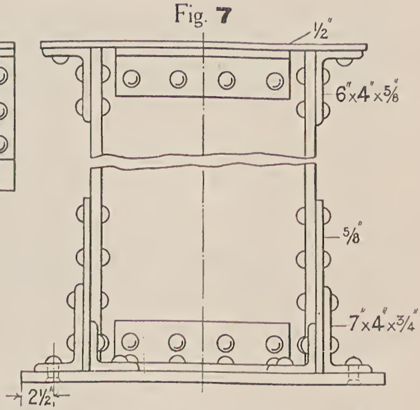


Fig. **7**



Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 67·3 tons (span: 18·1 feet). The "Web Cleats" by themselves are suited to a distributed load of 38·5 tons only (span: 31·7 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, 5·63 square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 19·2 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 28·9 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 172. Code Word: "ALBATROSS."
Nominal Dimensions 19″ × 12″ × 128 lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.		LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 92.		Broad Flange Beam(s) 19″ × 12″ approx. - H. J. SKELTON & Co.'s Section No. 172	128 lbs. per ft.
		N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	17½ ft. per ton
Figs. 1-3. Web Cleats	2	Angles 6″ × 4″ × ⅝″ by 14½″ long - -	48 lbs. per pair
	8	⅞″ Rivets, 2″ grip - - - - -	6½ lbs.
	8	⅞″ Bolts - - - - - - -	12 lbs. say
		Gross weight of Web Cleats	66½ lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1	Angle 4″ × 4″ × ⅝″ by 11¾″ long - -	15½ lbs.
	4	⅞″ Rivets, 1⅞″ grip - - - - -	3 lbs.
	4	⅞″ Bolts - - - - - - -	6 lbs. say
		Gross weight of one Upper Flange Cleat	24½ lbs.
Figs. 1, 2. Lower Flange Cleat	1	Angle 6″ × 4″ × ⅝″ by 11¾″ long - -	19½ lbs.
		Gross weight including 4 Bolts & 8 Rivets	32 lbs. say
Fig. 4. Fishplates	2	12″ × 14½″ × ½″ - - - - - -	49 lbs. per pair
	16	⅞″ Bolts, 3″ long - - - - -	20 lbs.
		Gross weight of Fishplates	69 lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2	Angles to flanges 6″ × 4″ × ⅝″ by 11¾″ long	39 lbs. per pair
	2	Angles to web 4″ × 4″ × ⅝″ by 14½″ long -	38 lbs. per pair
	1	Cover Plate (if required) 27″ × 12″ × ½″ -	46 lbs.
	22	⅞″ Rivets (2 countersunk) - - -	17½ lbs.
	(16)	⅞″ Bolts - - - - - - -	26 lbs. say
		Gross weight of one Light Cap (including plate)	166½ lbs.
Fig. 6. Stanchion Cap (heavy)	2	Angles to flanges 9″ × 4″ × ⅝″ by 11¾″ long	52 lbs. per pair
		Angles to web, Bolts and Plate as for Light Cap - - - - - - -	110 lbs.
	30	⅞″ Rivets (2 countersunk) - - -	24 lbs.
		Gross weight of one Heavy Cap (including plate)	186 lbs.
Figs. 5-8. Stanchion Base	1	Sole Plate 30″ × 27″ × ⅞″ - - - - -	201 lbs.
	2	Gusset Plates 27″ × 14″ × ⅝″ - - -	134 lbs. per pair
	2	Angles to flanges 7″ × 4″ × ¾″ by 27″ long	118 lbs. per pair
	2	Angles to web 4″ × 4″ × ⅝″ by 14½″ long -	38 lbs. per pair
	4	Angle Spreaders 4″ × 4″ × ⅝″ by 7½″ long -	39 lbs. total
	84	⅞″ Rivets (32 countersunk) - - -	63 lbs.
	4	Holding-down Bolts 1½″ × 24″ with 6″ × 6″ × ½″ plates - - - -	83 lbs.
		Gross weight of one Stanchion Base	676 lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 5 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

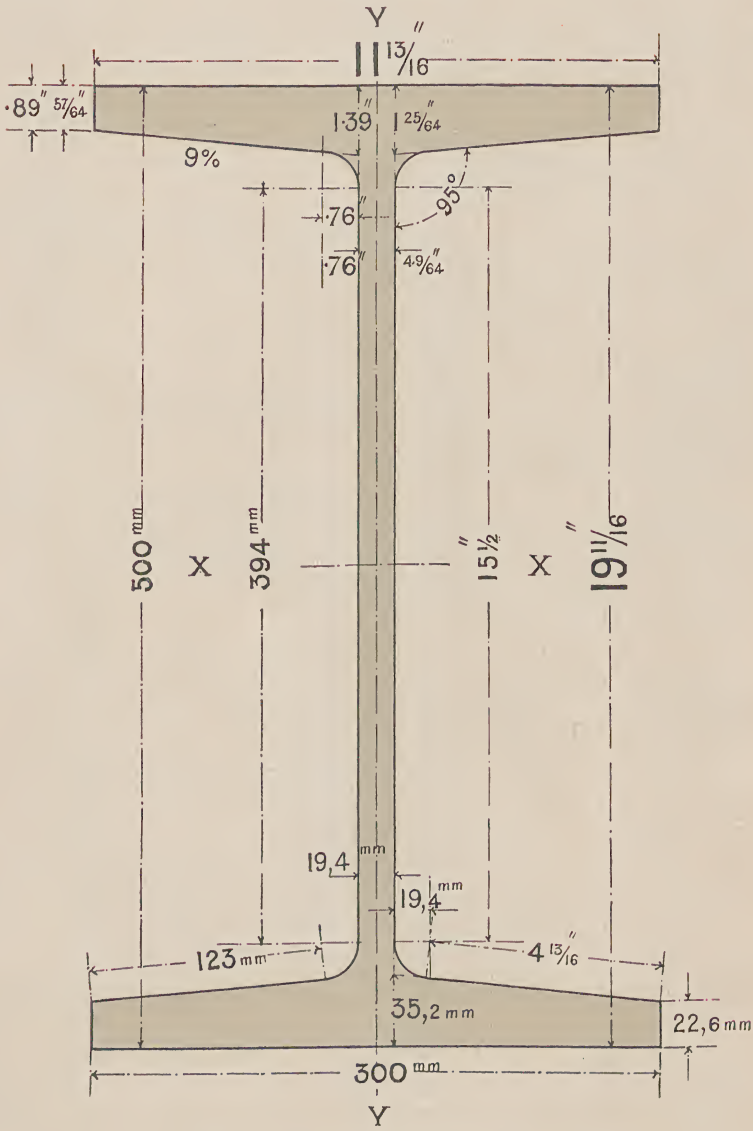
For further notes, see page 19.

SECTION DRAWING OF

B.F. BEAM { Section No. 176. Code Word: "ALCHEMY."
Nominal Dimensions 20″ × 12″ × 138 lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]

STOCK SIZE.



SCALE 1/4 (3 inches = 1 foot).

B.F. BEAM

PROPERTIES AND SAFE LOADS OF

Section No. 176. Code Word: "ALCHEMY."
Nominal Dimensions 20" ×12" ×138 lbs. per foot.
Actual Dimensions $19\frac{11}{16}'' \times 11\frac{13}{16}'' \times 138$ lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=16 feet to the ton, approx.) - -	138 lbs. per foot	205·5 kilos. per m.
Sectional Area - - - - - -	40·6 sq. ins.	261·8 sq. cm.
Greatest Moment of Inertia (Axis XX) - -	2674 inches^4	111283 cm^4
Least ,, ,, (Axis YY) - -	282 ,,	11718 ,,
Greatest Section Modulus (Axis XX) - -	272 inches^3	4451 cm^3
Least ,, ,, (Axis YY) - -	47·7 ,,	781 ,,
Moment of Resistance (XX) at $7\frac{1}{2}$ tons stress -	2037 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	125 tons (11 ft.)	
Greatest Radius of Gyration (Axis XX) - -	8·12 inches	
Least ,, ,, (Axis YY) - -	2·63 ,,	
Maximum length rolled - - - - -	about 68 feet	about 21 metres

The "MOMENT OF RESISTANCE" is the "Greatest Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: $7\frac{1}{2}$ tons per square inch).

Span (feet) - -	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons) -	...	*125*	113	97	85	76	68	57	48	42	38	34	31	26
Deflection (inches)	...	...	·13	·18	·23	·29	·36	·52	·70	·92	1·2	1·4	1·7	2·4

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about $22\frac{1}{2}$ inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·57 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet) -	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons) -	206	204	199	195	191	188	184	178	172	168	163	158	152	136
Nett Weight (cwts.)	9·86	11·1	12·3	13·6	14·8	16·0	17·3	18·5	19·7	20·9	22·2	23·4	24·6	27·1

Weight of Stanchion Base, about 7·42 cwts.; Cap, about 2·03 cwts.

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). For basement stanchions, add the weight of cap and base; for upper stanchions, add the weight of cap only. An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·47 tons if applied to one side of the flanges, or 1·32 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171).

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 176. Code Word: "ALCHEMY."
Nominal Dimensions 20″×12″×138 lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

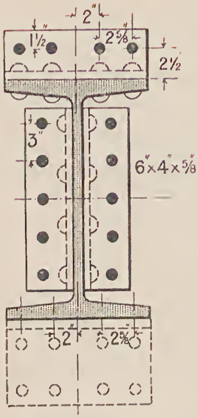


Fig. 1

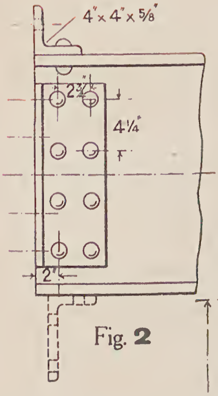


Fig. 2

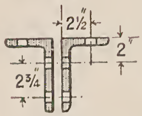


Fig. 3

SCALE 1/16

($\frac{3}{4}$ inch = 1 foot)

Rivets and Bolts $\frac{7}{8}$″

Bolt-Holes $\frac{15}{16}$″

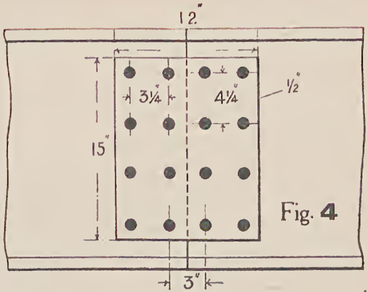


Fig. 4

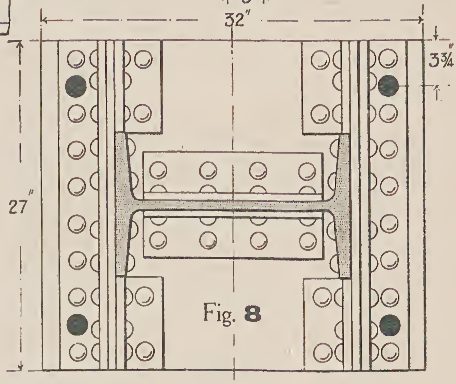


Fig. 8

Fig. 5

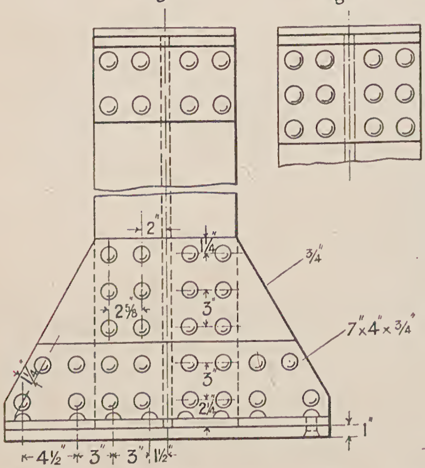


Fig. 6

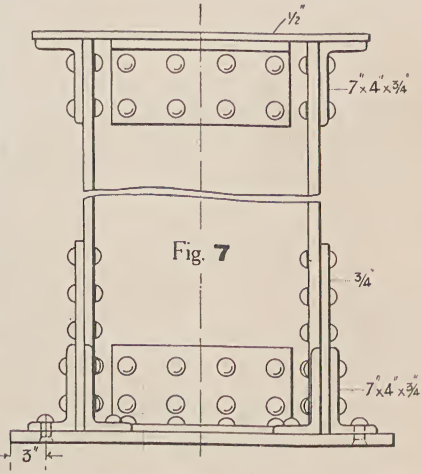


Fig. 7

Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 74·6 tons (span: 18·2 feet). The "Web Cleats" by themselves are suited to a distributed load of 48·1 tons only (span: 28·2 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Figs. 5-8. Stanchion, Cap (2 designs) and Base. Area of Base or Sole Plate, 6 square feet. Safe Load on each Flange Cleat in light cap (Fig. 5), 19·2 tons. Safe Load on each Flange Cleat in heavy cap (Fig. 6), 28·9 tons. For further explanation, see page 16.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 176. Code Word: "ALCHEMY."
Nominal Dimensions 20″×12″×138 lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.		LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 96.		Broad Flange Beam(s) 20″×12″ approx. - H. J. SKELTON & Co.'s Section No. 176	138 lbs. per ft.
		N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	16 ft. per ton
Figs. 1-3. Web Cleats	2	Angles 6″×4″×$\frac{5}{8}$″ by 15″ long - -	50 lbs. per pair
	8	$\frac{7}{8}$″ Rivets, 2″ grip - - - - -	$6\frac{1}{2}$ lbs.
	10	$\frac{7}{8}$″ Bolts - - - - - - -	15 lbs. say
		Gross weight of Web Cleats	$71\frac{1}{2}$ lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1	Angle 4″×4″×$\frac{5}{8}$″ by $11\frac{3}{4}$″ long - -	$15\frac{1}{2}$ lbs.
	4	$\frac{7}{8}$″ Rivets, $1\frac{7}{8}$″ grip - - - - -	3 lbs.
	4	$\frac{7}{8}$″ Bolts - - - - - - -	6 lbs. say
		Gross weight of one Upper Flange Cleat	$24\frac{1}{2}$ lbs.
Figs. 1, 2. Lower Flange Cleat	1	Angle 7″×4″×$\frac{3}{4}$″ by $11\frac{3}{4}$″ long - -	26 lbs.
		Gross weight including 4 Bolts & 8 Rivets	39 lbs. say
Fig. 4. Fishplates	2	12″×15″×$\frac{1}{2}$″ - - - - - - -	51 lbs. per pair
	16	$\frac{7}{8}$″ Bolts, 3″ long - - - - -	20 lbs.
		Gross weight of Fishplates	71 lbs. per pair
Figs. 5, 7. Stanchion Cap (light)	2	Angles to flanges 7″×4″×$\frac{3}{4}$″ by $11\frac{3}{4}$″ long	51 lbs. per pair
	2	Angles to web 7″×4″×$\frac{3}{4}$″ by 15″ long -	65 lbs. per pair
	1	Cover Plate (if required) 28″×12″×$\frac{1}{2}$″ -	48 lbs.
	26	$\frac{7}{8}$″ Rivets (2 countersunk) - - -	21 lbs.
	(16)	$\frac{7}{8}$″ Bolts - - - - - - -	26 lbs. say
		Gross weight of one Light Cap (including plate)	211 lbs.
Fig. 6. Stanchion Cap (heavy)	2	Angles to flanges 9″×4″×$\frac{3}{4}$″ by $11\frac{3}{4}$″ long	61 lbs. per pair
		Angles to web, Bolts and Plate as for Light Cap - - - - - -	139 lbs.
	34	$\frac{7}{8}$″ Rivets (2 countersunk) - - -	27 lbs.
		Gross weight of one Heavy Cap (including plate)	227 lbs.
Figs. 5-8. Stanchion Base	1	Sole Plate 32″×27″×1″ - - - -	245 lbs.
	2	Gusset Plates 27″×$15\frac{1}{2}$″×$\frac{3}{4}$″ - - -	178 lbs. per pair
	2	Angles to flanges 7″×4″×$\frac{3}{4}$″ by 27″ long -	118 lbs. per pair
	2	Angles to web 7″×4″×$\frac{3}{4}$″ by 15″ long -	65 lbs. per pair
	4	Angle Spreaders 7″×4″×$\frac{3}{4}$″ by $7\frac{1}{2}$″ long -	65 lbs. total
	96	$\frac{7}{8}$″ Rivets (32 countersunk) - - -	77 lbs.
	4	Holding-down Bolts $1\frac{1}{2}$″×24″ with 6″×6″×$\frac{1}{2}$″ plates - - -	83 lbs.
		Gross weight of one Stanchion Base	831 lbs.

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 5 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste. The lengths given in the above table for plates and angles cut slantwise are the lengths of the longer side.

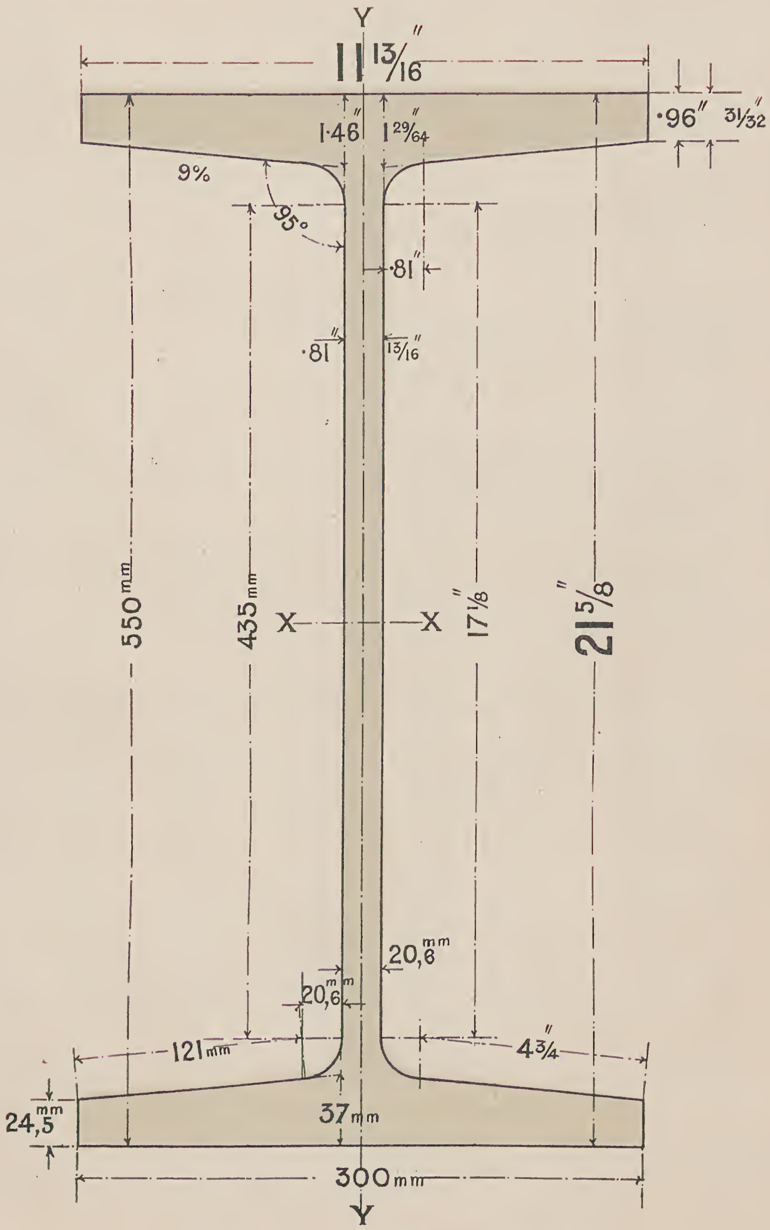
The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

For further notes, see page 19.

SECTION DRAWING OF

B.F. BEAM { Section No. 180. Code Word: "ALCOHOL." Nominal Dimensions 22″×12″×152 lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]



SCALE 1/4 (3 inches=1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM { Section No. 180. Code Word: "ALCOHOL."
Nominal Dimensions 22″ × 12″ × 152 lbs. per foot.
Actual Dimensions $21\frac{5}{8}$″ × $11\frac{13}{16}$″ × 152 lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=15 feet to the ton, approx.)	152 lbs. per foot	226·1 kilos. per m.
Sectional Area	44·7 sq. ins.	288·0 sq. cm.
Greatest Moment of Inertia (Axis XX)	3509 inches^4	145957 cm^4
Least ,, ,, (Axis YY)	302 ,,	12582 ,,
Greatest Section Modulus (Axis XX)	324 inches^3	5308 cm^3
Least ,, ,, (Axis YY)	51·2 ,,	839 ,,
Moment of Resistance (XX) at $7\frac{1}{2}$ tons stress	2429 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	142 tons (11 ft.)	
Greatest Radius of Gyration (Axis XX)	8·86 inches	
Least ,, ,, (Axis YY)	2·60 ,,	
Maximum length rolled	about 60 feet	about 18-20 metres

The "MOMENT OF RESISTANCE" is the "Greatest Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: $7\frac{1}{2}$ tons per square inch).

Span (feet)	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons)	...	*142*	135	116	101	90	81	67	58	51	45	40	37	31
Deflection (inches)	...	...	·12	·16	·21	·26	·33	·47	·64	·83	1·1	1·3	1·6	2·2

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. Deflections to the right of the black vertical line exceed 1/300th of the span. The wall-bearing for this section should be about 27 inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·62 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet)	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons)	226	223	218	213	209	206	200	194	188	184	178	172	166	149
Nett Weight (cwts.)	10·8	12·2	13·6	14·9	16·3	17·6	19·0	20·3	21·7	23·0	24·4	25·8	27·1	29·8

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·49 tons if applied to one of the flanges, or 1·35 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171). Two beams of this section, placed side by side, make an excellent and economical stanchion for exceptionally heavy loads. (See illustrations on page 181, "Foundations.")

22″
24″
26″
30″

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 180. Code Word: "ALCOHOL."
Nominal Dimensions 22" × 12" × 152 lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

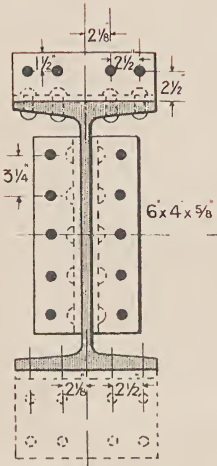


Fig. 1

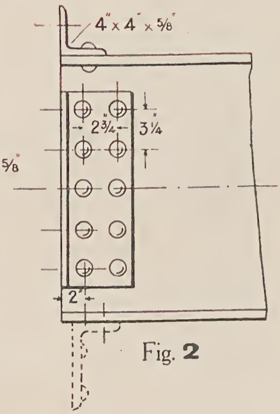


Fig. 2

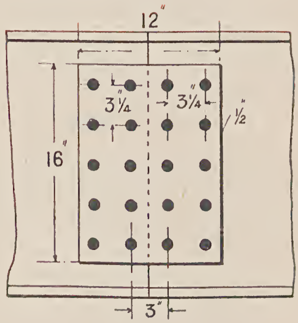


Fig. 4

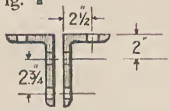


Fig. 3

Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 74·6 tons (span: 21·7 feet). The "Web Cleats" by themselves are suited to a distributed load of 48·1 tons only (span: 33·7 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Flange Plates. For carrying heavy loads over a long span, the strength of this section may be doubled by suitable flange plates without risk of overstraining the web. See table of safe loads on plated beams (page 185) and accompanying notes.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 180. Code Word: "ALCOHOL."
Nominal Dimensions 22″×12″×152 lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.		LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 100.		Broad Flange Beam(s) 22″×12″ approx. -	152 lbs. per ft.
		*H. J. SKELTON & Co.'s Section No. 180	
		N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	15 ft. per ton
Figs. 1-3. Web Cleats	2	Angles 6″×4″×$\frac{5}{8}$″ by 16″ long - -	53 lbs. per pair
	10	$\frac{7}{8}$″ Rivets, $2\frac{1}{8}$″ grip - - - - - -	$8\frac{1}{2}$ lbs.
	10	1″ Bolts - - - - - - -	22 lbs. say
		Gross weight of Web Cleats	$83\frac{1}{2}$ lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1	Angle 4″×4″×$\frac{5}{8}$″ by $11\frac{3}{4}$″ long - -	$15\frac{1}{2}$ lbs.
	4	$\frac{7}{8}$″ Rivets, 2″ grip - - - -	3 lbs.
	4	$\frac{7}{8}$″ or 1″ Bolts - - - - - -	9 lbs. say
		Gross weight of one Upper Flange Cleat	$27\frac{1}{2}$ lbs.
Figs. 1, 2. Lower Flange Cleat	1	Angle 7″×4″×$\frac{3}{4}$″ by $11\frac{3}{4}$″ long - -	26 lbs.
		Gross weight including 4 Bolts & 8 Rivets	42 lbs. say
Fig. 4. Fishplates	2	12″×16″×$\frac{1}{2}$″ - - - - - - -	$54\frac{1}{2}$ lbs. per pair
	20	1″ Bolts, 3″ long - - - - -	35 lbs.
		Gross weight of Fishplates	$89\frac{1}{2}$ lbs. per pair

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 5 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste.

The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

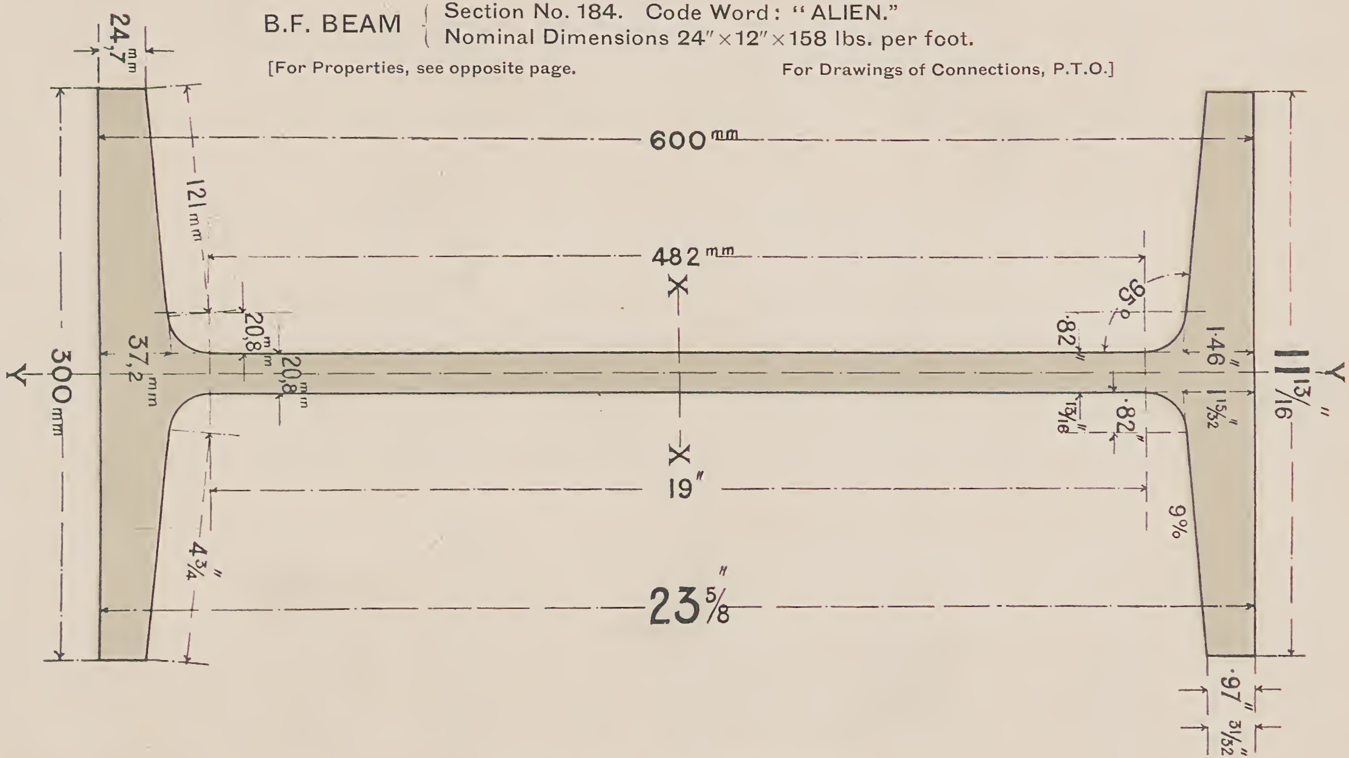
For further notes, see page 19.

24″
26″
30″

SECTION DRAWING OF

B.F. BEAM { Section No. 184. Code Word: "ALIEN."
Nominal Dimensions 24″×12″×158 lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM — Section No. 184. Code Word: "ALIEN."
Nominal Dimensions 24" × 12" × 158 lbs. per foot.
Actual Dimensions $23\frac{5}{8}$" × $11\frac{13}{16}$" × 158 lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=14 feet to the ton, approx.)	158 lbs. per foot	236·0 kilos. per m.
Sectional Area	46·6 sq. ins.	300·6 sq. cm.
Greatest Moment of Inertia (Axis XX)	4308 inches^4	179303 cm^4
Least " " (Axis YY)	305 "	12672 "
Greatest Section Modulus (Axis XX)	365 inches^3	5977 cm^3
Least " " (Axis YY)	51·6 "	845 "
Moment of Resistance (XX) at $7\frac{1}{2}$ tons stress	2738 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	141 tons (13 ft.)	
Greatest Radius of Gyration (Axis XX)	9·62 inches	
Least " " (Axis YY)	2·56 "	
Maximum length rolled	about 60 feet	about 18-20 metres

The "Moment of Resistance" is the "Greatest Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: $7\frac{1}{2}$ tons per square inch).

Span (feet)	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons)	...	...	*141*	130	114	101	91	76	65	57	50	45	41	35
Deflection (inches)	...	...	...	·15	·19	·24	·30	·43	·59	·77	·97	1·2	1·5	2·0

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. None of the tabulated deflections exceeds 1/300th of the span. The wall-bearing for this section should be about 27 inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·68 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet)	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons)	236	233	227	222	218	214	207	202	195	190	185	178	170	153
Nett Weight (cwts.)	11·3	12·7	14·2	15·6	17·0	18·4	19·8	21·2	22·6	24·1	25·5	26·9	28·3	31·1

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·51 tons if applied to one of the flanges, or 1·37 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171). Two beams of this section, placed side by side, make an excellent and economical stanchion for exceptionally heavy loads. (See illustrations on page 181, "Foundations.")

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 184. Code Word: "ALIEN." Nominal Dimensions 24″×12″×158 lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

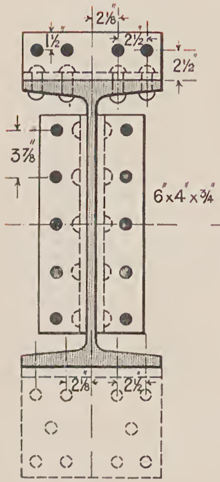


Fig. 1

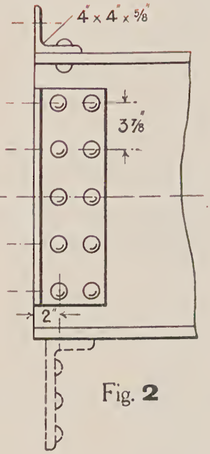


Fig. 2

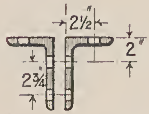


Fig. 3

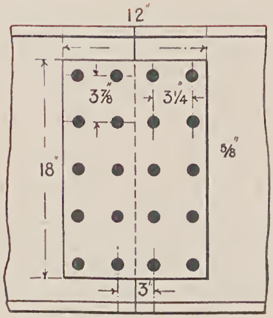


Fig. 4

Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 84·2 tons (span: 21·7 feet). The "Web Cleats" by themselves are suited to a distributed load of 48·1 tons only (span: 38 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Flange Plates. For carrying heavy loads over a long span, the strength of this section may be doubled by suitable flange plates without risk of overstraining the web. See table of safe loads on plated beams (page 185) and accompanying notes.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 184. Code Word : "ALIEN." Nominal Dimensions 24″×12″×158 lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.	LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 104.	Broad Flange Beam(s) 24″×12″ approx. -	158 lbs. per ft.
	H. J. SKELTON & Co.'s Section No. 184	
	N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	14 ft. per ton
Figs. 1-3. Web Cleats	2 Angles 6″×4″×$\frac{3}{4}$″ by 18″ long - -	71 lbs. per pair
	10 $\frac{7}{8}$″ Rivets, $2\frac{3}{8}$″ grip - - - - -	9 lbs.
	10 1″ Bolts - - - - - - -	22 lbs. say
	Gross weight of Web Cleats	102 lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1 Angle 4″×4″×$\frac{5}{8}$″ by $11\frac{3}{4}$″ long - -	$15\frac{1}{2}$ lbs.
	4 $\frac{7}{8}$″ Rivets, 2″ grip - - - - -	3 lbs.
	4 $\frac{7}{8}$″ or 1″ Bolts - - - - - -	9 lbs. say
	Gross weight of one Upper Flange Cleat	$27\frac{1}{2}$ lbs.
Figs. 1, 2. Lower Flange Cleat	1 Angle 9″×4″×$\frac{3}{4}$″ by $11\frac{3}{4}$″ long - -	31 lbs.
	Gross weight including 4 Bolts & 10 Rivets	49 lbs. say
Fig. 4. Fishplates	2 12″×18″×$\frac{5}{8}$″ - - - - - -	77 lbs. per pair
	20 1″ Bolts, $3\frac{1}{2}$″ long - - - - -	37 lbs.
	Gross weight of Fishplates	114 lbs. per pair

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 5 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste.

The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

For further notes, see page 19.

26″

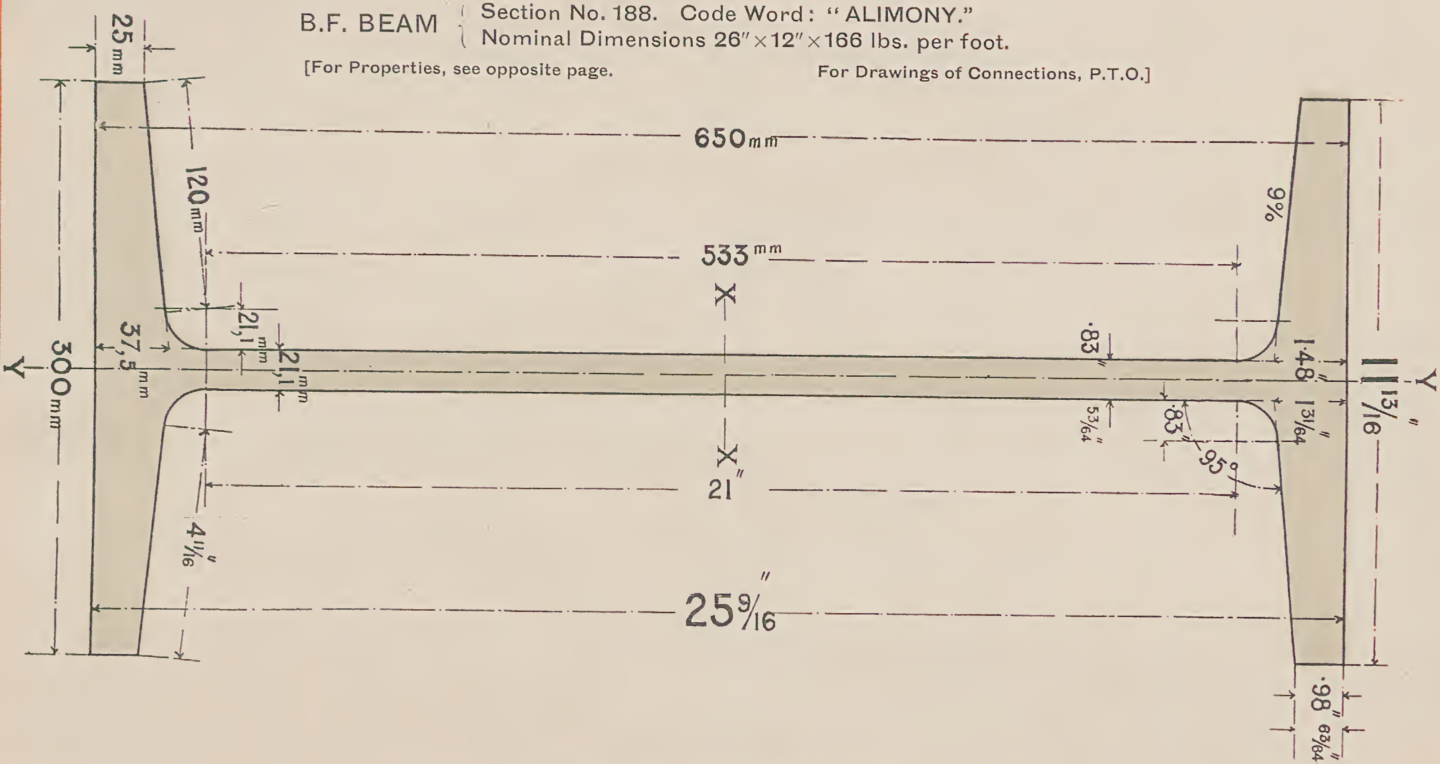
30″

SECTION DRAWING OF

B.F. BEAM { Section No. 188. Code Word: "ALIMONY." Nominal Dimensions 26″×12″×166 lbs. per foot.

[For Properties, see opposite page.]

For Drawings of Connections, P.T.O.]



SCALE 1/4 (3 inches=1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM — Section No. 188. Code Word: "ALIMONY."
Nominal Dimensions 26" × 12" × 166 lbs. per foot.
Actual Dimensions $25\frac{9}{16}$" × $11\frac{13}{16}$" × 166 lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=$13\frac{1}{2}$ feet to the ton, approx.)	166 lbs. per foot	246·9 kilos. per m.
Sectional Area	48·7 sq. ins.	314·5 sq. cm.
Greatest Moment of Inertia (Axis XX)	5224 inches^4	217402 cm^4
Least ,, ,, (Axis YY)	308 ,,	12814 ,,
Greatest Section Modulus (Axis XX)	408 inches^3	6690 cm^3
Least ,, ,, (Axis YY)	52·1 ,,	854 ,,
Moment of Resistance (XX) at $7\frac{1}{2}$ tons stress	3062 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	141 tons (15 ft.)	
Greatest Radius of Gyration (Axis XX)	10·4 inches	
Least ,, ,, (Axis YY)	2·51 ,,	
Maximum length rolled	52-56 feet	16-17 metres

The "MOMENT OF RESISTANCE" is the "Greatest Section Modulus" multiplied by the working stress (here taken as $7\frac{1}{2}$ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: $7\frac{1}{2}$ tons per square inch).

Span (feet)	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons)	...	...	...	*141*	127	113	102	84	72	63	56	51	46	39
Deflection (inches)	...	...	...	...	·18	·23	·28	·40	·54	·71	·90	1·1	1·3	1·9

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. None of the tabulated deflections exceeds 1/300th of the span. The wall-bearing for this section should be about 27 inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·68 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

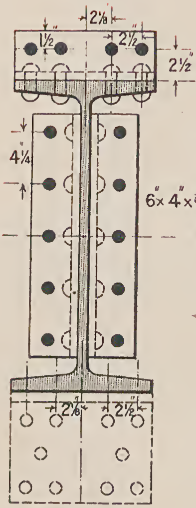
Height (feet)	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons)	247	242	237	232	227	222	215	209	202	197	190	183	176	157
Nett Weight (cwts.)	11·8	13·3	14·8	16·3	17·8	19·2	20·7	22·2	23·7	25·2	26·6	28·1	29·6	32·6

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·53 tons if applied to one of the flanges, or 1·39 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171). Two beams of this section, placed side by side, make an excellent and economical stanchion for exceptionally heavy loads. (See illustrations on page 181, "Foundations.")

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 188. Code Word: "ALIMONY."
Nominal Dimensions 26" × 12" × 166 lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]



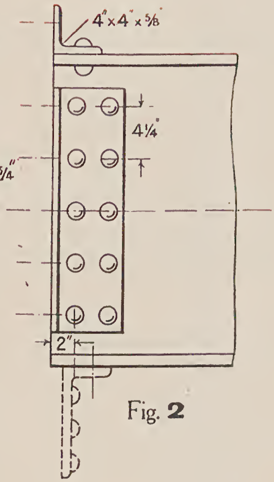


Fig. **2**

Fig. **1**

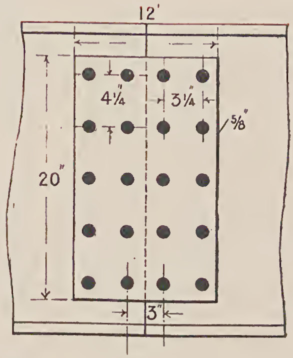


Fig. **4**

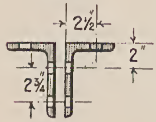


Fig. **3**

Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 84·2 tons (span: 24·3 feet). The "Web Cleats" by themselves are suited to a distributed load of 48·1 tons only (span: 42·4 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Flange Plates. For carrying heavy loads over a long span, the strength of this section may be doubled by suitable flange plates without risk of overstraining the web. See table of safe loads on plated beams (page 185) and accompanying notes.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 188. Code Word: "ALIMONY."
Nominal Dimensions 26″ × 12″ × 166 lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.		LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 108.		Broad Flange Beam(s) 26″ × 12″ approx. -	166 lbs. per ft.
		H. J. SKELTON & Co.'s Section No. 188	
		N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey-Mill," see page 225.	13½ ft. per ton
Figs. 1-3. Web Cleats	2	Angles 6″ × 4″ × ¾″ by 20″ long - -	79 lbs. per pair
	10	⅞″ Rivets, 2⅜″ grip - - - - -	9 lbs.
	10	1″ Bolts - - - - - -	22 lbs. say
		Gross weight of Web Cleats	110 lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1	Angle 4″ × 4″ × ⅝″ by 11¾″ long - -	15½ lbs.
	4	⅞″ Rivets, 2″ grip - - - - -	3 lbs.
	4	⅞″ or 1″ Bolts - - - - - -	9 lbs. say
		Gross weight of one Upper Flange Cleat	27½ lbs.
Figs. 1, 2. Lower Flange Cleat	1	Angle 9″ × 4″ × ¾″ by 11¾″ long - -	31 lbs.
		Gross weight including 4 Bolts & 10 Rivets	49 lbs. say
Fig. 4. Fishplates	2	12″ × 20″ × ⅝″ - - - - - -	85 lbs. per pair
	20	1″ Bolts, 3½″ long - - - - -	37 lbs.
		Gross weight of Fishplates	122 lbs. per pair

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 5 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste.

The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

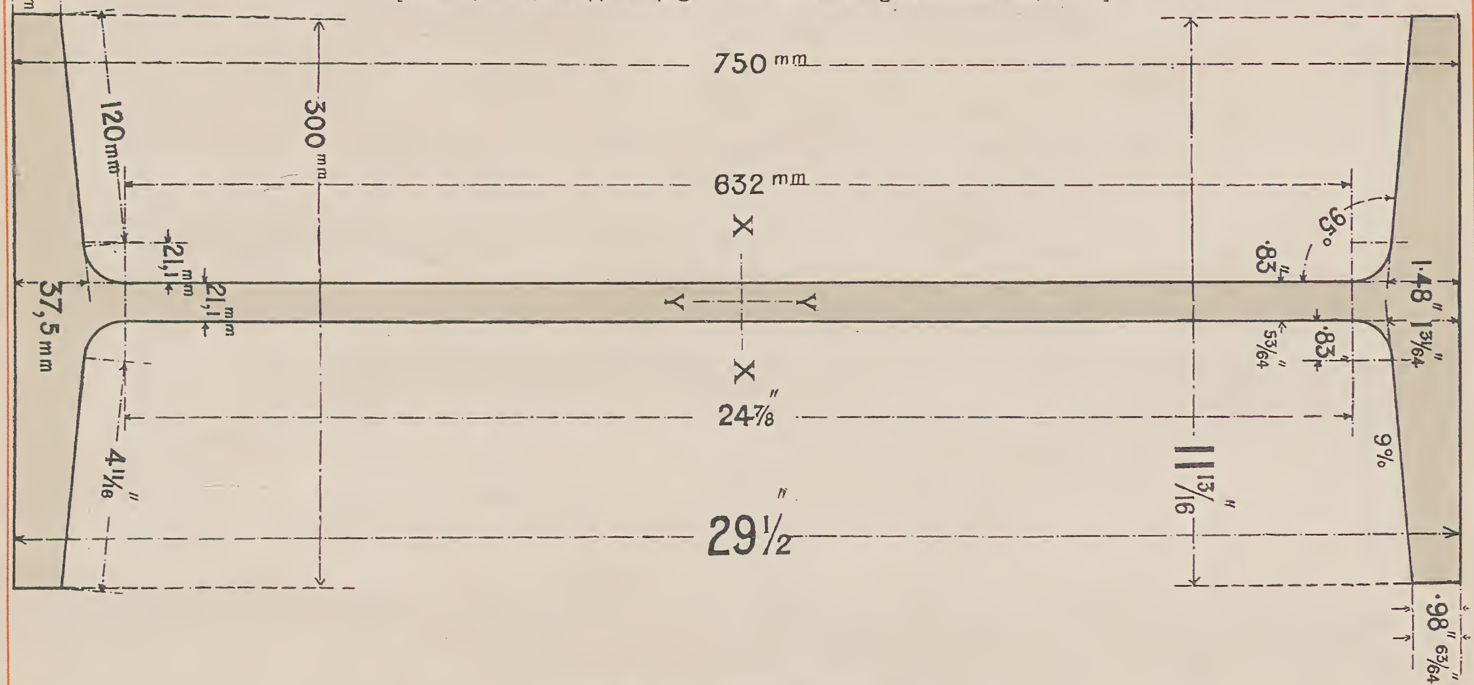
For further notes, see page 19.

30″

SECTION DRAWING OF

B.F. BEAM { Section No. 196. Code Word: "ALKALI."
Nominal Dimensions 30″ × 12″ × 177 lbs. per foot.

[For Properties, see opposite page. For Drawings of Connections, P.T.O.]



SCALE 1/4 (3 inches = 1 foot).

PROPERTIES AND SAFE LOADS OF

B.F. BEAM
- Section No. 196. Code Word: "ALKALI."
- Nominal Dimensions 30" × 12" × 177 lbs. per foot.
- Actual Dimensions 29½" × 11 13/16" × 177 lbs. per foot.

[See Drawing opposite.]

PROPERTIES IN BRITISH AND METRIC UNITS.

Weight (=12½ feet to the ton, approx.)	177 lbs. per foot	263·4 kilos. per m.
Sectional Area	52·0 sq. ins.	335·7 sq. cm.
Greatest Moment of Inertia (Axis XX)	7271 inches⁴	302560 cm⁴
Least " " (Axis YY)	308 "	12823 "
Greatest Section Modulus (Axis XX)	492 inches³	8068 cm³
Least " " (Axis YY)	52·2 "	855 "
Moment of Resistance (XX) at 7½ tons stress	3692 ton-inches	
Max. Safe Distributed Load, without "stiffeners"	126 tons (20 ft.)	
Greatest Radius of Gyration (Axis XX)	11·8 inches	
Least " " (Axis YY)	2·43 "	
Maximum length rolled	52-56 feet	16-17 metres

The "MOMENT OF RESISTANCE" is the "Greatest Section Modulus" multiplied by the working stress (here taken as 7½ tons per square inch). When a beam is loaded irregularly, ascertain the maximum bending-moment (ton-inches) and find a beam of which the "Moment of Resistance" is equal to the ascertained bending-moment. The safe distributed load (tons) which a beam will carry (theoretically) on a span of 1 foot is two-thirds of the "Moment of Resistance" (ton-inches).

SAFE DISTRIBUTED LOADS Etc. (Working Stress: 7½ tons per square inch.)

Span (feet)	8	10	12	14	16	18	20	24	28	32	36	40	44	52
Safe Load (tons)	...	...	...	...	...	*126*	123	102	87	76	68	61	55	47
Deflection (inches)	...	...	...	...	...	...	·24	·34	·47	·61	·77	·96	1·2	1·6

The above Safe Loads and Deflections are extracted from the general table of safe loads on page 120, where the methods of calculation are fully explained. None of the tabulated deflections exceeds 1/300th of the span. The wall-bearing for this section should be about 27 inches at each end. [For Stone Templates etc., see page 126.] The weight of the beam (in cwts.) can be ascertained by reference to the table for stanchions below. If the beam is connected to stanchions, the end-fastenings will weigh about 0·68 cwt. per end.

SAFE LOADS for above Section used as a STANCHION.

Height (feet)	8	9	10	11	12	13	14	15	16	17	18	19	20	22
Safe Load (tons)	263	256	251	246	241	234	226	219	213	206	198	191	178	163
Nett Weight (cwts.)	12·6	14·2	15·8	17·4	19·0	20·5	22·1	23·7	25·3	26·9	28·4	30·0	31·6	34·8

The above Safe Loads are extracted from Table A of safe loads on stanchions, page 156, and are specially adapted for ordinary building work. The "weights" given in the table are the nett calculated weights (cwts.). An eccentric or unbalanced load of 1 ton, on a stanchion of this section, is equivalent to a central load of 2·56 tons if applied to one of the flanges, or 1·42 tons if applied to one side of the web (see chapter on "Eccentrically Loaded Stanchions," page 171). Two beams of this section, placed side by side, make an excellent and economical stanchion for exceptionally heavy loads. (See illustrations on page 181, "Foundations.")

30"

DRAWINGS OF CONNECTIONS SUITED TO

B.F. BEAM { Section No. 196. Code Word: "ALKALI."
Nominal Dimensions 30″×12″×177 lbs. per foot.

[For Properties, see previous page. For Weights of Materials, see opposite page.]

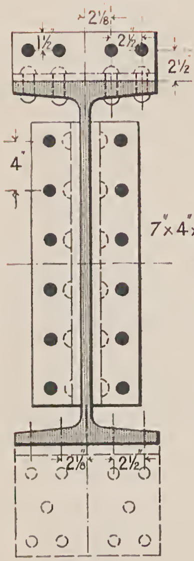


Fig. 1

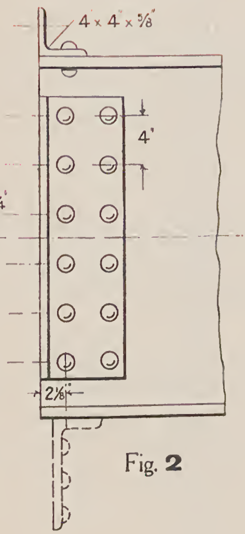


Fig. 2

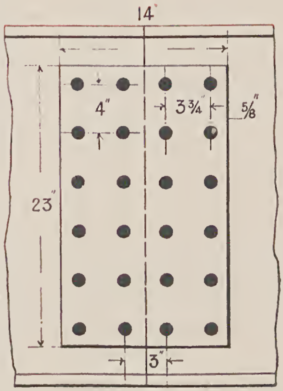


Fig. 4

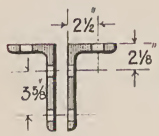


Fig. 3

Figs. 1-3. End Connection for a Girder. The combined "Web and Flange Cleats" are suited to a distributed load of about 91·4 tons (span: 26·9 feet). The "Web Cleats" by themselves are suited to a distributed load of 57·7 tons only (span: 42·6 feet). For further explanation, see page 13.

Fig. 4. Fishplates. For further explanation, see page 15.

Flange Plates. For carrying heavy loads over a long span, the strength of this section may be doubled by suitable flange plates without risk of overstraining the web. See table of safe loads on plated beams (page 185) and accompanying notes.

MATERIALS REQUIRED FOR CONNECTIONS TO

B.F. BEAM { Section No. 196. Code Word: "ALKALI." Nominal Dimensions 30″×12″×177 lbs. per foot.

[See Drawings on opposite page.]

No. and Description of Drawing.		LIST OF MATERIALS.	Approximate Weight.
Section Drawing on page 112.		Broad Flange Beam(s) 30″ × 12″ approx. -	177 lbs. per ft.
		H. J. SKELTON & Co.'s Section No. 196	
		N.B.—The general specification should stipulate that "All Broad Flange Beams must be rolled in a Grey Mill," see page 225.	$12\frac{1}{2}$ ft. per ton
Figs. 1-3. Web Cleats	2	Angles 7″ × 4″ × $\frac{3}{4}$″ by 23″ long - -	100 lbs. per pair
	12	$\frac{7}{8}$″ Rivets, $2\frac{3}{8}$″ grip - - - - -	11 lbs.
	12	1″ Bolts - - - - - - -	26 lbs. say
		Gross weight of Web Cleats	137 lbs. per pair
Figs. 1, 2. Upper Flange Cleat	1	Angle 4″ × 4″ × $\frac{5}{8}$″ by $11\frac{3}{4}$″ long - -	$15\frac{1}{2}$ lbs.
	4	$\frac{7}{8}$″ Rivets, 2″ grip - - - - -	3 lbs.
	4	$\frac{7}{8}$″ or 1″ Bolts - - - - - -	9 lbs. say
		Gross weight of one Upper Flange Cleat	$27\frac{1}{2}$ lbs.
Figs. 1, 2. Lower Flange Cleat	1	Angle 9″ × 4″ × $\frac{3}{4}$″ by $11\frac{3}{4}$″ long - -	31 lbs.
		Gross weight including 4 Bolts & 10 Rivets	49 lbs. say
Fig. 4. Fishplates	2	14″ × 23″ × $\frac{5}{8}$″ - - - - - -	114 lbs. per pair
	24	1″ Bolts, $3\frac{1}{2}$″ long - - - - -	45 lbs.
		Gross weight of Fishplates	159 lbs. per pair

The weights of the above materials are *theoretic* weights and are subject to a rolling margin of 5 % under or over.

In calculating the weights of cleats, plates etc., no deduction has been made for bolt-holes and similar waste.

The stated lengths of bolts are to be measured from under head to point. It is assumed that taper washers will be used where nuts or bolt-heads bed on the inner surface of flanges.

For further notes, see page 19.

CONTENTS OF PART II.

CONTENTS OF PART II.—Continued.

MISCELLANEOUS ILLUSTRATIONS.

The above photograph shows Broad Flange Beams as Stanchions in the Works of Messrs. Gimson & Co., Ltd., Engineers, Leicester.

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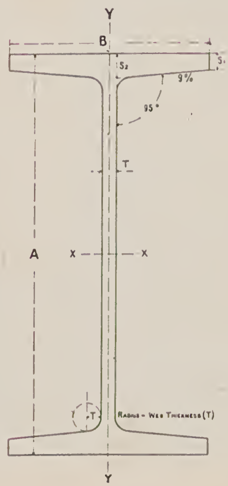
BROAD FLANGE BEAMS.

LIST OF SIZES.

Nominal Dimensions. A	Nominal Dimensions. B	Exact Dimensions. A	Exact Dimensions. B	Weight per foot.	Reference No. and Code Word.		Web. T	Flanges. S1	Flanges. S2
Inches.		Inches.		Lbs.			Ins.	Ins.	Ins.
7	× 7	$7\frac{1}{8}$	× $7\frac{1}{8}$	$31\frac{1}{2}$	104	Abbess	·33	·35	·66
8	× 8	$7\frac{7}{8}$	× $7\frac{7}{8}$	37	108	Abode	·33	·37	·71
$8\frac{1}{2}$	× $8\frac{1}{2}$	$8\frac{11}{16}$	× $8\frac{11}{16}$	44	112	Abstainer	·35	·39	·77
$9\frac{1}{2}$	× $9\frac{1}{2}$	$9\frac{7}{16}$	× $9\frac{7}{16}$	51	116	Abyss	·39	·41	·82
10	× 10	$9\frac{7}{8}$	× $9\frac{7}{8}$	55	120	Accent	·41	·43	·85
$10\frac{1}{4}$	× $10\frac{1}{4}$	$10\frac{1}{4}$	× $10\frac{1}{4}$	61	124	Acolyte	·43	·46	·90
$10\frac{1}{2}$	× $10\frac{1}{2}$	$10\frac{5}{8}$	× $10\frac{5}{8}$	65	128	Actor	·44	·47	·93
11	× 11	11	× 11	70	132	Actuary	·45	·49	·96
$11\frac{1}{2}$	× $11\frac{1}{2}$	$11\frac{7}{16}$	× $11\frac{7}{16}$	75	136	Adamant	·47	·50	·99
12	× 12	$11\frac{13}{16}$	× $11\frac{13}{16}$	80	140	Adder	·49	·52	1·03
$12\frac{1}{2}$	× 12	$12\frac{5}{8}$	× $11\frac{13}{16}$	85	144	Adept	·51	·56	1·06
$13\frac{1}{2}$	× 12	$13\frac{3}{8}$	× $11\frac{13}{16}$	88	148	Admirer	·53	·57	1·08
14	× 12	$14\frac{3}{16}$	× $11\frac{13}{16}$	96	152	Advent	·56	·64	1·14
15	× 12	$14\frac{15}{16}$	× $11\frac{13}{16}$	101	156	Aggressor	·58	·67	1·17
16	× 12	$15\frac{3}{4}$	× $11\frac{13}{16}$	107	160	Agnostic	·61	·72	1·22
17	× 12	$16\frac{3}{4}$	× $11\frac{13}{16}$	113	164	Alabaster	·63	·75	1·25
18	× 12	$17\frac{11}{16}$	× $11\frac{13}{16}$	121	168	Alarmist	·67	·80	1·31
19	× 12	$18\frac{11}{16}$	× $11\frac{13}{16}$	128	172	Albatross	·69	·84	1·34
20	× 12	$19\frac{11}{16}$	× $11\frac{13}{16}$	138	176	Alchemy	·76	·89	1·39
22	× 12	$21\frac{5}{8}$	× $11\frac{13}{16}$	152	180	Alcohol	·81	·96	1·46
24	× 12	$23\frac{5}{8}$	× $11\frac{13}{16}$	158	184	Alien	·82	·97	1·46
26	× 12	$25\frac{9}{16}$	× $11\frac{13}{16}$	166	188	Alimony	·83	·98	1·48
30	× 12	$29\frac{1}{2}$	× $11\frac{13}{16}$	177	196	Alkali	·83	·98	1·48



BROAD FLANGE BEAMS.

PROPERTIES.

Nominal Dimensions.	Sectional Area.	Greatest Moment of Resistance.	Moments of Inertia.		Section Moduli.		Radii of Gyration.	
			XX	YY	XX	YY	XX	YY
Inches.	Sq. Ins.	Ton-inches.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.
7 × 7	9·3	179	84·4	25·8	23·8	7·27	3·01	1·67
8 × 8 *	10·9	237	124	37·7	31·6	9·58	3·38	1·86
8½ × 8½*	12·8	307	177	53·3	41·0	12·3	3·72	2·04
9½ × 9½*	15·0	391	247	73·1	52·2	15·5	4·05	2·21
10 × 10 *	16·3	442	290	85·9	58·9	17·5	4·22	2·30
10¼ × 10¼	17·9	505	345	102	67·4	20·0	4·38	2·39
10½ × 10½	19·1	560	397	118	74·7	22·3	4·56	2·49
11 × 11 *	20·4	623	458	136	83·1	24·7	4·73	2·58
11½ × 11½	21·9	690	526	154	92·0	27·0	4·90	2·66
12 × 12 *	23·6	769	605	180	103	30·5	5·07	2·76
12½ × 12	24·9	862	724	189	115	32·0	5·39	2·75
13½ × 12	25·9	949	847	195	127	33·0	5·71	2·74
14 × 12 *	28·1	1080	1021	211	144	35·8	6·02	2·74
15 × 12	29·6	1192	1190	221	159	37·4	6·33	2·73
16 × 12 *	31·6	1324	1390	234	177	39·6	6·63	2·72
17 × 12	33·2	1470	1640	242	196	41·0	7·03	2·70
18 × 12 *	35·6	1646	1944	256	219	43·4	7·39	2·68
19 × 12	37·5	1827	2278	268	244	45·3	7·79	2·67
20 × 12 *	40·6	2037	2674	282	272	47·7	8·12	2·63
22 × 12	44·7	2429	3509	302	324	51·2	8·86	2·60
24 × 12	46·6	2738	4308	305	365	51·6	9·62	2·56
26 × 12	48·7	3062	5224	308	408	52·1	10·4	2·51
30 × 12	52·0	3692	7271	308	492	52·2	11·8	2·43

EXPLANATION OF THE ABOVE TABLE.

CODE WORDS Etc.

In ordering Broad Flange Beams or other rolled steel shapes given in this book, it is sufficient to quote the corresponding reference number *or* the telegraphic code word (see page 220, "*Code*"). It is desirable to stipulate that "Broad Flange Beams must be rolled in a Grey Mill" (see page 225, "*Grey Mill*").

LEAST RADIUS OF GYRATION.

The ratios of length or height to least radius of gyration are tabulated on page 175 for Broad Flange Beams of various lengths.

MOMENTS OF RESISTANCE.

The figures tabulated in this column are calculated as follows:—Moment of Resistance = Section Modulus (xx) multiplied by 7½ tons per square inch.

To find what size of beam is required to carry an irregular load, ascertain the maximum bending-moment (ton-inches). Then select whichever of the above sections has a "Moment of Resistance" equal to the ascertained bending-moment.

N.B.—The various properties tabulated above are defined on page 8.

* Sizes marked with an asterisk are "Stock Sizes," *i.e.* are freely and regularly stocked at works and by various girder merchants etc.

BROAD FLANGE BEAMS.

TABLE OF SAFE UNIFORMLY DISTRIBUTED LOADS (TONS) AND CORRESPONDING DEFLECTIONS IN INCHES.

Size.	Weight per foot.	6 ft.		8 ft.		10 ft.		12 ft.		14 ft.		16 ft.		18 ft.		20 ft.		22 ft.		24 ft.	
		Safe Load.	Deflect'n.	Safe Load.	Deflect'n.	Safe Load.	Deflect'n.	Safe Load.	Deflect'n.	Safe Load.	Deflect'n.	Safe Load.	Deflect'n.	Safe Load.	Deflect'n.	Safe Load.	Deflect'n.	Safe Load.	Deflect'n.	Safe Load.	Deflect'n.
Inches.	Lbs.	Tns.	Ins.	Tns.	Ins.	Tns.	Ins.	Tns.	Ins.	Tns.	Ins.	Tns.	Ins.	Tns.	Ins.	Tns.	Ins.	Tns.	Ins.	Tns.	Ins.
7 × 7	$31\frac{1}{2}$	20	·09	15	·16	12	·25	10	·36	8	·49	7	·64	6	·80	6	·99	...	...	...	...
8 × 8	37	*25*	...	19	·14	15	·22	13	·32	11	·44	10	·57	9	·73	8	·90	...	...	...	...
$8\frac{1}{2} \times 8\frac{1}{2}$	44	*28*	...	26	·13	20	·20	17	·29	15	·40	13	·52	11	·66	10	·81	9	·98	9	1·2
$9\frac{1}{2} \times 9\frac{1}{2}$	51	*35*	...	33	·12	26	·19	22	·27	19	·37	16	·48	14	·61	13	·75	12	·93	11	1·1
10 × 10	55	*38*	...	37	·11	29	·18	25	·26	21	·35	18	·46	16	·58	15	·72	13	·87	12	1·0
$10\frac{1}{4} \times 10\frac{1}{4}$	61	*42*	...	42	·11	34	·17	28	·25	24	·34	21	·44	19	·56	17	·69	15	·83	14	1·0
$10\frac{1}{2} \times 10\frac{1}{2}$	65	...	...	*44*	...	37	·17	31	·24	27	·32	23	·42	21	·54	19	·67	17	·80	16	·96
11 × 11	70	...	...	*47*	...	42	·16	35	·23	30	·31	26	·41	23	·52	21	·64	19	·77	17	·92
$11\frac{1}{2} \times 11\frac{1}{2}$	75	...	...	*51*	...	46	·15	38	·22	33	·30	29	·40	25	·50	23	·62	21	·75	19	·90
12 × 12	80	...	...	*55*	...	51	·15	43	·22	37	·29	32	·39	28	·49	26	·60	23	·72	21	·86
$12\frac{1}{2} \times 12$	85	...	...	*59*	...	58	·14	48	·20	41	·27	36	·36	32	·45	29	·56	26	·68	24	·81
$13\frac{1}{2} \times 12$	88	...	...	*63*	...	63	·13	53	·19	45	·26	40	·34	35	·43	32	·53	29	·64	26	·76
14 × 12	96	...	...	...	...	*71*	...	60	·18	51	·24	45	·32	40	·40	36	·50	33	·60	30	·72
15 × 12	101	...	...	...	...	*74*	...	66	·17	57	·23	50	·30	44	·38	40	·47	36	·57	33	·68
16 × 12	107	...	...	...	...	*82*	...	73	·16	63	·22	55	·29	49	·36	44	·45	40	·54	37	·65
17 × 12	113	...	...	...	...	*87*	...	82	·15	70	·21	61	·27	54	·34	49	·42	45	·51	41	·61
18 × 12	121	...	...	...	...	*99*	...	92	·14	78	·20	69	·25	61	·32	55	·40	50	·48	46	·57
19 × 12	128	...	...	...	...	*103*	...	101	·14	87	·19	76	·24	68	·31	61	·38	55	·46	51	·54
20 × 12	138	...	...	...	...	*125*	...	113	·13	97	·18	85	·23	76	·29	68	·36	62	·43	57	·52
22 × 12	152	...	...	...	...	*142*	...	135	·12	116	·16	101	·21	90	·26	81	·33	74	·39	67	·47
24 × 12	158	...	...	...	...	...	...	*141*	...	130	·15	114	·19	101	·24	91	·30	83	·36	76	·43
26 × 12	166	...	...	...	...	...	...	...	...	*141*	...	127	·18	113	·23	102	·28	93	·34	84	·40
30 × 12	177	...	...	...	...	...	...	...	...	...	...	...	...	*126*	...	123	·24	111	·29	102	·34

denote that the beam
the web buckling

NOTES TO THE ABOVE TABLE.

SAFE LOADS.

(1) The safe loads are calculated for a "working stress" of $7\frac{1}{2}$ tons per square inch by the usual formula for a beam supported at both ends, namely :—

Safe Load = 8 × Section Modulus (XX) × Working Stress ÷ Span (inches).

(2) If the tabular loads are reduced, the working stress will be reduced in the same proportion.

MOVING LOADS.

The tabulated loads are suitable for practically stationary loads as in ordinary buildings. For moving loads, as in bridges, it is usual to multiply the "live load" by $1\frac{1}{2}$ to 2 (according to speed etc.) to obtain the equivalent stationary load. Girders in factories, when subject to slight shock and vibration from machinery, should generally be proportioned to carry about $1\frac{1}{2}$ times the actual load.

CONCENTRATED LOADS Etc.

For loads concentrated at the centre of the beam, multiply the actual load by 2; that is, allow 50% only of the tabular load.

The table on page 124 shows what proportions of the tabular loads may be taken for a few simple examples of irregular loading.

FLOOR GIRDERS.

For selecting floor girders, the table on page 142 will usually be found more convenient.

GIRDERS CARRYING BRICK WALLS. See special table overleaf and notes on page 128.

BEAMS WITHOUT SIDE SUPPORT.

If the beam is without side support and the span exceeds 20 times the flange width, see page 141.

SHEARING STRESSES IN BEAMS. See page 140.

BROAD FLANGE BEAMS.—Continued.

TABLE OF SAFE UNIFORMLY DISTRIBUTED LOADS (TONS) AND CORRESPONDING DEFLECTIONS IN INCHES.

Size.	Weight per foot.	26 ft.		28 ft.		30 ft.		32 ft.		34 ft.		36 ft.		38 ft.		40 ft.		44 ft.		48 ft.	
		Safe Load.	Deflect'n.	Safe Load.	Deflect'n.	Safe Load.	Deflect'n.	Safe Load.	Deflect'n.	Safe Load.	Deflect'n.	Safe Load.	Deflect'n.	Safe Load.	Deflect'n.	Safe Load.	Deflect'n.	Safe Load.	Deflect'n.	Safe Load.	Deflect'n.
Inches.	Lbs.	Tns.	Ins.	Tns.	Ins.	Tns.	Ins.	Tns.	Ins.	Tns.	Ins.	Tns.	Ins.	Tns.	Ins.	Tns.	Ins.	Tns.	Ins.	Tns.	Ins.
7 × 7	31½	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
8 × 8	37	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
8½ × 8½	44	8	1·4	7	1·6	7	1·8	...	...	...	...	...	...	...	...	...	...	...	...	...	...
9½ × 9½	51	10	1·3	9	1·5	9	1·7	8	1·9	...	...	...	...	...	...	...	...	...	...	...	...
10 × 10	55	11	1·2	11	1·4	10	1·6	9	1·8	...	...	...	...	...	...	...	...	...	...	...	...
10¼ × 10¼	61	13	1·2	12	1·4	11	1·6	10	1·8	10	2·0	...	...	...	...	...	...	...	...	...	...
10½ × 10½	65	14	1·1	13	1·3	12	1·5	12	1·7	11	1·9	10	2·2	...	...	...	...	...	...	...	...
11 × 11	70	16	1·1	15	1·3	14	1·4	13	1·6	12	1·9	11	2·1	11	2·3	...	...	...	...	...	...
11½ × 11½	75	18	1·0	16	1·2	15	1·4	14	1·6	13	1·8	13	2·0	12	2·2	11	2·5	...	...	...	...
12 × 12	80	20	1·0	18	1·2	17	1·3	16	1·5	15	1·7	14	1·9	13	2·2	13	2·4	...	...	...	...
12½ × 12	85	22	·95	21	1·1	19	1·3	18	1·4	17	1·6	16	1·8	15	2·0	14	2·2	...	...	...	...
13½ × 12	88	24	·90	23	1·0	21	1·2	20	1·4	19	1·5	18	1·7	17	1·9	16	2·1	14	2·6	...	...
14 × 12	96	28	·84	26	·98	24	1·1	23	1·3	21	1·4	20	1·6	19	1·8	18	2·0	16	2·4	15	2·9
15 × 12	101	31	·80	28	·93	27	1·1	25	1·2	23	1·4	22	1·5	21	1·7	20	1·9	18	2·3	17	2·7
16 × 12	107	34	·76	31	·88	29	1·0	28	1·2	26	1·3	25	1·4	23	1·6	22	1·8	20	2·2	18	2·6
17 × 12	113	38	·71	35	·83	33	·95	31	1·1	29	1·2	27	1·4	26	1·5	24	1·7	22	2·0	20	2·4
18 × 12	121	42	·67	39	·78	37	·90	34	1·0	32	1·2	30	1·3	29	1·4	27	1·6	25	1·9	23	2·3
19 × 12	128	47	·64	43	·74	41	·85	38	·97	36	1·1	34	1·2	32	1·4	30	1·5	28	1·8	25	2·2
20 × 12	138	52	·61	48	·70	45	·81	42	·92	40	1·0	38	1·2	36	1·3	34	1·4	31	1·7	28	2·1
22 × 12	152	62	·56	58	·64	54	·73	51	·83	48	·94	45	1·1	43	1·2	40	1·3	37	1·6	34	1·9
24 × 12	158	70	·51	65	·59	61	·67	57	·77	53	·85	50	·97	48	1·1	45	1·2	41	1·5	38	1·7
26 × 12	166	78	·47	72	·54	67	·62	63	·71	60	·80	56	·90	53	1·0	51	1·1	46	1·3	42	1·6
30 × 12	177	95	·40	87	·47	81	·54	76	·61	72	·69	68	·77	65	·86	61	·96	55	1·2	51	1·4

NOTES TO THE ABOVE TABLE.—Continued.

SAFE LOADS PRINTED IN ITALICS.

The safe loads printed in italics are calculated in the manner explained on page 140 ("Shearing Stresses in Beams").

TEMPLATES Etc. FOR SUPPORTING ENDS OF GIRDERS. See page 126.

DEFLECTIONS.

(1) These are calculated by the usual formula (see page 137) for a beam supported at both ends and uniformly loaded. If the actual load is less than the tabular load, the deflection will be less, in exactly the same proportion.

(2) Deflections to the right of the zig-zag line exceed 1/300th of the span, which is not desirable.

(3) But, spans to the right of the zig-zag line can be used without excessive deflection if the tabular loads are suitably reduced (*vide* special table overleaf, also table on page 142, "Floor Girders").

(4) In calculating the safe loads, it is proper to assume that the ends are merely supported. The tabulated deflections are also calculated on this assumption. But, the actual deflection of a Broad Flange Beam, especially in the case of the smaller sections, will probably be considerably less than the tabular deflection (see page 138). [The deflection of a continuous girder or beam with "fixed ends" would be theoretically one-fifth of the tabular deflection.]

N.B.—For more detailed notes on "Deflection," see pages 137 etc.

AMERICAN READERS may find the following rules useful:—

(1) To ascertain the safe distributed loads in tons of 2,000 lbs. for an "extreme fiber stress" of 16,000 lbs. per square inch:—*Increase the tabular loads by one-fifteenth (or reduce the actual load by one-sixteenth).*

(2) To ascertain the safe distributed loads in tons of 2,000 lbs. for an "extreme fiber stress" of 12,500 lbs. per square inch:—*Reduce the tabular loads by one-sixth (or increase the actual load by one-fifth).*

ADVANTAGES OF WIDE-FLANGED GIRDERS. See overleaf.

TABLE OF SAFE UNIFORMLY DISTRIBUTED LOADS ON BROAD FLANGE BEAMS, REDUCED FOR A MAXIMUM DEFLECTION OF 1/500th OF THE SPAN.

Size. A	Size. B	Weight per foot.	Safe Distributed Loads (tons) for Spans of 12 to 56 feet. 12′	14′	16′	18′	20′	24′	28′	32′	36′	40′	44′	48′	52′	56′
Inches.		Lbs.														
7	× 7	31½	8·0	5·9	4·50	3·56	2·88	2·00	1·47	1·13	...	...	...	...	...	...
8	× 8	37	11·8	8·6	6·6	5·2	4·23	2·94	2·16	1·65	1·31	...	...	...	...	...
8½	× 8½	44	17	12·3	9·4	7·5	6·0	4·20	3·08	2·36	1·86	1·51	...	...	...	...
9½	× 9½	51	22	17·2	13·2	10·4	8·4	5·9	4·30	3·29	2·60	2·11	1·74	...	...	...
10	×10	55	25	20	15·5	12·2	9·9	6·9	5·0	3·87	3·05	2·48	2·04	1·72	...	...
10¼	×10¼	61	28	24	18·4	14·5	11·8	8·2	6·0	4·60	3·63	2·94	2·43	2·04	...	...
10½	×10½	65	31	27	21	16·7	13·5	9·4	6·9	5·3	4·18	3·39	2·80	2·35	...	...
11	×11	70	35	30	24	19·3	15·6	10·9	8·0	6·1	4·82	3·91	3·23	2·71	2·31	...
11½	×11½	75	38	33	28	22	18·0	12·5	9·2	7·0	5·5	4·49	3·71	3·12	2·66	...
12	×12	80	43	37	32	25	21	14·3	10·5	8·1	6·4	5·2	4·27	3·59	3·06	2·63
12½	×12	85	48	41	36	30	25	17·2	12·6	9·6	7·6	6·2	5·1	4·29	3·66	3·15
13½	×12	88	53	45	40	35	29	20	14·8	11·3	8·9	7·2	6·0	5·0	4·28	3·69
14	×12	96	60	51	45	40	35	24	17·8	13·6	10·8	8·7	7·2	6·0	5·1	4·44
15	×12	101	66	57	50	44	40	28	21	15·9	12·5	10·2	8·4	7·0	6·0	5·2
16	×12	107	73	63	55	49	44	33	24	18·5	14·6	11·9	9·8	8·2	7·0	6·0
17	×12	113	82	70	61	54	49	39	29	22	17·3	14·0	11·6	9·7	8·3	7·1
18	×12	121	92	78	69	61	55	46	34	26	20	16·6	13·7	11·5	9·8	8·5
19	×12	128	101	87	76	68	61	51	40	30	24	19·4	16·1	13·5	11·5	9·9
20	×12	138	113	97	85	76	68	57	47	36	28	23	18·9	15·9	13·5	11·6
22	×12	152	135	116	101	90	81	67	58	47	37	30	25	21	17·7	15·3
24	×12	158	141	130	114	101	91	76	65	57	45	37	30	25	22	18·8
26	×12	166	...	141	127	113	102	84	72	63	55	45	37	31	26	23
30	×12	177	...	...	...	126	123	102	87	76	68	61	51	43	37	32
Deflection at Centre of Beam (inches).			·29″	·34″	·38″	·43″	·48″	·58″	·67″	·77″	·86″	·96″	1·06″	1·15″	1·25″	1·34″

USES OF THE ABOVE TABLE.

It is suggested that the above table should be used in selecting girders to carry brick walls (rules for estimating the weight of brickwork are given on page 128): also for selecting floor beams in cases where it is desired to use shallow beams over unusually long spans without exceeding the permissible deflection, and in all other cases where small loads have to be carried over comparatively long spans or where deflection is the governing consideration.

EXPLANATION OF THE ABOVE TABLE.

(1) The formula by which the safe loads printed in *large* type were calculated, is stated on page 139. The deflection will be 1/500th of the span. Working stress: 7½ tons per square inch or less.

(2) The safe loads printed in *small* type are copied from the previous table. Calculated deflection: 1/500th of span or less. Working stress: 7½ tons per square inch.

(3) The "Deflections at Centre" as given at the foot of the table, represent 1/500th of the span (inches).

(4) If both ends of the beam are rigidly fixed, the deflection under the tabular load will be only 1/2500th of the span.

(5) For a load concentrated at the centre of the beam, take 5/8ths of the tabular load, provided that the safe load thus calculated does not exceed one-half of the safe load given in the previous table.

(6) To calculate the loads corresponding to deflections other than 1/500th of the span:—increase or reduce the tabular loads in the same proportion; taking care, however, not to exceed the safe loads given in the previous table.

(7) The tabular loads include the weight of the beam itself.

(8) If the beam is over 17 inches deep, is without side support and the span exceeds 20 feet, see page 141 ("Beams without Side Support").

ADVANTAGES OF WIDE-FLANGED GIRDERS.

(1) Headroom can be saved. For example, a $10\frac{1}{4}''$ Broad Flange Beam will carry the same load as an ordinary $14''$ joist. It is, of course, most economical in metal to employ as deep a girder as possible, but headroom is sometimes of more importance. In some cases, the saving in brickwork alone may compensate for the extra weight of metal involved in using shallow girders.

(2) The required weight of girders to carry a floor in any given depth is less for Broad Flange Beams than for any other type of girder obtainable. [See further notes on this subject on pages 132 and 136.]

(3) As compared with all forms of riveted girders of corresponding dimensions, Broad Flange Beams offer the advantages of reduced weight and cost, simplicity and saving of time in erection etc. [See further notes on this subject, page 136.]

(4) Broad Flange Beams can be used over long spans in cases where intermediate supports are undesirable but where they would be necessary if ordinary rolled steel joists were used.

(5) The lateral stiffness of a beam increases as the square of its flange width. Hence Broad Flange Beams can be used without lateral support over much longer spans than ordinary joists of nominally equal strength. In this respect, Broad Flange Beams are eminently suitable for use as crane girders etc. [See illustration of crane-bearing girders and stanchions on page 129.]

(6) Broad Flange Beams offer striking advantages for use as main girders in concrete floors (see chapter on "Floor Girders"), as cantilevers, raking beams etc. in theatres and churches (see illustration on page 155), and numerous other special applications.

(7) Broad Flange Beams make excellent soleplates for tramrail joints (see illustrations on page 239), and are useful as templates and rail-bearing girders of all kinds.

(8) The advantages of wide-flanged girders are particularly apparent when used in conjunction with wide-flanged stanchions in self-supporting steel structures of all kinds, such as steel-framed buildings etc. The standard fastenings and connections to Broad Flange Beams are far stronger, although simpler, than the best practicable fastenings to narrow-flanged material (compound girders or ordinary joists). Consequently, wide-flanged girders stiffen the stanchions to which they are attached, and provide end-fixing of far greater value than can possibly be obtained in using ordinary joists or riveted girders. In this way, wide-flanged girders contribute greatly to the lateral stability and general rigidity of any structure in which they are employed.

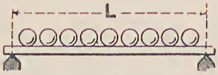
(9) When Broad Flange Beams are freely and intelligently applied in any heavy steel structure, as in the coal staith illustrated on page 234, it is impossible to design a similar structure with narrow-flanged material at so low a cost, on account of the saving in weight afforded by Broad Flange Beams and the greatly diminished labour bill in the shops and in erection at site. [See introduction to Part I., pages 10 to 12.]

(10) Broad Flange Beams are better rolled and more regular in quality than ordinary joists. [See notes on the "Grey Mill," page 222.]

(11) Broad Flange Beams are less expensive in maintenance and less liable to rust than built-girders.

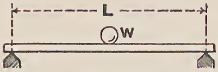
SAFE LOADS, DEFLECTIONS Etc. OF STEEL GIRDERS FOR SOME CASES OF SPECIAL LOADING.

[L = Span in inches.]



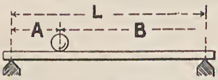
Supported at both ends.
Uniformly Loaded.

Total Safe Load (W tons) = Tabular Load.
Maximum Deflection (inches) = Tabular Deflection.
Maximum Bending-Moment (ton-inches) = W L $\div$ 8 at centre.
Maximum Shear (tons) - = $\frac{1}{2}$ W at each support.



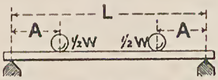
Supported at both ends.
Load concentrated at centre.

Safe Load (W) - - = One-half of the Tabular Load.
Maximum Deflection - = Four-fifths of the Tabular Deflection.
Maximum Bending-Moment = W L $\div$ 4 at centre.
Maximum Shear - - = $\frac{1}{2}$ W at each support.



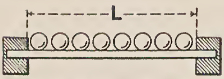
Supported at both ends.
Load concentrated but not at centre.

Safe Load (W) - - = Tabular Load $\times$ L^2 $\div$ 8. AB.
Maximum Deflection - = Less than 4/5ths of the Tabular Deflection.
Maximum Bending-Moment = W A B $\div$ L under Load.
Maximum Shear - - = W B $\div$ L at support near A.



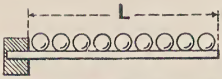
Supported at both ends.
Two Symmetrical Loads.

Total Safe Load (W) - = Tabular Load $\times$ L $\div$ 4 A.
Maximum Deflection - = Approximately equal to the Tabular Deflection.
Maximum Bending-Moment = $\frac{1}{2}$ W A at centre.
Maximum Shear - - = $\frac{1}{2}$ W at each support.



Fixed at both ends (or continuous).
Uniformly Loaded.

Total Safe Load (W) - = $1\frac{1}{2}$ times the Tabular Load.
Maximum Deflection - = Three-tenths of the Tabular Deflection.
Maximum Bending-Moment = W L $\div$ 12 at each support.
Maximum Shear - - = $\frac{1}{2}$ W at each support.



Fixed at one end (cantilever).
Uniformly Loaded.

Total Safe Load (W) - = One-fourth of the Tabular Load.
Maximum Deflection - = Tabular Deflection $\times$ 2·4.
Maximum Bending-Moment = $\frac{1}{2}$ W L at support.
Maximum Shear - - = W at support.

(1) The term "fixed ends" implies that the end-fixing would remain intact even if the beam were loaded to destruction. Such conditions rarely, if ever, occur in building construction. [See also foot-note on page 138.]

(2) A continuous beam is one which is laid in a single length over several supports. It is important to remember that a slight subsidence of one of the supports may seriously increase the stresses. It is necessary also that the load on each span should be a permanent one, correctly proportioned to the span.

(3) The simplest way to ascertain the required size of beam to carry an irregular load is to calculate the maximum bending-moment (ton-inches, *not* ton-feet). Then refer to the table of properties of Broad Flange Beams on page 119, and select a size of which the tabulated "Moment of Resistance" is equal to the ascertained bending-moment. Or, divide the ascertained bending-moment by the working stress to be allowed (by $7\frac{1}{2}$ tons per square inch, say). The result gives the required "Section Modulus."

(4) In the formulæ given above, the weight of the beam itself is neglected.

(5) To ascertain the maximum shearing stress in web (tons per square inch) divide the "Maximum Shear" (tons) by D $\times$ T, where D = depth of beam (inches) and T = web thickness (inches).

COMPOUND OR RIVETED GIRDERS COMPOSED OF BROAD FLANGE BEAMS.

It is needless to say that many of the advantages of Broad Flange Beams disappear when compounded etc. At the same time, they retain certain advantages over other built-girders :—

(1) Higher carrying power in proportion to weight.

(2) Greater flange area available for connections.

Properties and safe loads for a series of Broad Flange Beams with flange plates are given on page 184.

Girders consisting of two Broad Flange Beams (with or without flange plates) make economical and efficient substitutes for extra heavy compound girders, consisting of two or three ordinary rolled steel joists with plates riveted to the flanges. The latter combination, namely three joists with flange plates, is often used to make girders 24″ wide to carry heavy walls. This same width can be obtained by combining any two Broad Flange Beams from 12″ to 30″ deep, and the saving in weight and cost thus effected is usually remarkable. The following is a typical example.

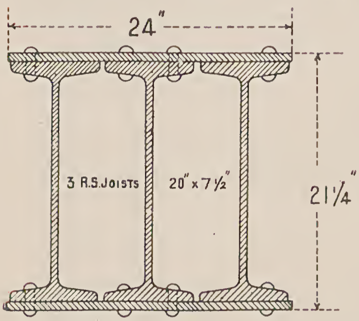


FIG. 1.

Section Modulus ... 728
Weight per foot ... 373 lbs.

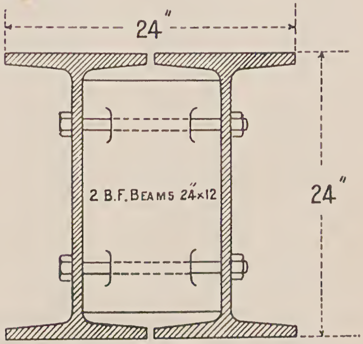


FIG. 2.

Section Modulus ... 730
Weight per foot ... 322 lbs.*

The Broad Flange Beams have the same strength as the Compound in Fig. 1 which weighs 16 % more. In addition to this, Fig. 1 is much more expensive on account of the drilling and riveting required. It should be noticed that the beams and separators for Fig. 2 can be forwarded separately and joined up on arrival at the building site. This is an obvious convenience and saves time. Fig. 2 has the disadvantage of being rather deeper. If, however, this slight extra depth happened to be prohibitive, it could be suitably avoided by employing two shallower Broad Flange Beams with plates riveted to the flanges. In this way the required strength could be obtained in 21 inches or less, though of course the cost would be greater than that of the plain beams and separators.†

* The above weight per foot includes an allowance of 6 lbs. per foot for cast-iron separators and bolts and nuts.

† There is an interesting example of a compound girder composed of Broad Flange Beams at the Conservatoire National, Paris. The girder in question is a conspicuous part of the huge 300-ton testing machine constructed for the French Government by Messrs. Joshua Buckton & Co., Ltd., of Leeds (J. H. Wicksteed, Esq., M.Inst.C.E.). This girder consists of two 30″ × 12″ Broad Flange Beams with plates riveted to the flanges and is 20 feet long.

MISCELLANEOUS NOTES ON GIRDERS.

BEARING PLATES AND BRICK PIERS.

BEARING PLATES FOR GIRDERS.

With narrow-flanged beams it is sometimes necessary to rivet bearing plates on the lower flange at the ends, to obtain sufficient bearing area upon the material upon which the beam rests, without unduly extending the length. In the case of broad-flanged beams this is never required, as the wider flange provides sufficient bearing area by itself when stone templates are used. The area of stone template should be proportioned according to the rules laid down in the section on "Foundations" (page 176), but if the thickness is less than one-fourth of the least width, a greater factor of safety should be taken.

BRICK PIERS.

When loads are carried by brick piers, the normal pressure should be reduced whenever the height of the pier exceeds six times its least width. A simple formula for the reduced pressure is $w = W\left(\frac{24-r}{18}\right)$ where w=allowable load per square foot, W=normal load per square foot according to nature of material, r=height divided by least diameter, in the same terms, feet or inches. Thus where 4 tons per foot super could be allowed on brickwork in mortar on a pier 3′ by 2′ 3″ and 13′ 6″ high, if the height were increased to 27′ the load should be reduced to $4\left(\frac{24-\frac{27}{2\cdot25}}{18}\right) = 2\frac{2}{3}$ tons per square foot.

CARRYING BEAM ENDS OVER OPENINGS.

It is frequently required to carry the end of a beam or girder over an opening by means of a rolled steel joist. Broad-flanged sections are particularly suitable for this purpose owing to their great strength combined with shallow depth. In selecting a beam for this purpose, it is needless to say that the load must be treated as a *concentrated* load.

BEAMS SERVING AS TEMPLATES.

Broad Flange Beams offer the same advantage of relatively shallow depth when used as templates to distribute the load transmitted by a girder end.

The template beam may be treated as a pair of cantilevers, each sustaining a distributed load equal to one-half of the total load. On this assumption, the beam must be of the same size as if the load were distributed over a span equal to its entire length. Consequently, the beam need only have one-half of the carrying power which would be required to carry a similar load over an opening.

In many cases, however, it is desirable to proportion the beam as for an opening: for example, if the ends of the template beam rest on piers and the intermediate portion on a light wall.

The strength of the web of the template beam directly under the girder may have to be considered, though it will rarely, if ever, happen that stiffeners are required. The point may be decided as follows:—If the girder supported on the template beam is B inches wide and carries a distributed load of W tons, then the load on the template is $\frac{1}{2}$W tons, and may be regarded from this point of view as spread over B lineal inches of the web of the template beam.

The loads per lineal inch and per lineal foot which the webs of Broad Flange Beams will safely sustain as columns, are given in the following table. If the load on the template ($\frac{1}{2}$W) divided by the width of the girder (B inches) exceeds the "safe direct load per lineal inch," stiffeners may be employed, or two shallower beams placed side by side.

A slight excess over the nominal safe load on the web may be disregarded, as the above mode of procedure assumes that the whole of the pressure is concentrated on that part of the web directly under the load, which is not the case.

BEAMS SERVING AS TEMPLATES Etc.

Section.	Safe Direct Load on Web. Per Lineal Inch.	Safe Direct Load on Web. Per Lineal Foot.	Section.	Safe Direct Load on Web. Per Lineal Inch.	Safe Direct Load on Web. Per Lineal Foot.	Section.	Safe Direct Load on Web. Per Lineal Inch.	Safe Direct Load on Web. Per Lineal Foot.
Inches.	Tons.	Tons.	Inches.	Tons.	Tons.	Inches.	Tons.	Tons.
7 × 7	1·94	23·3	$11\frac{1}{2} \times 11\frac{1}{2}$	2·64	31·8	18 × 12	3·58	42·9
8 × 8	1·88	22·6	12 × 12	2·78	33·5	19 × 12	3·61	43·4
$8\frac{1}{2} \times 8\frac{1}{2}$	1·96	23·5	$12\frac{1}{2} \times 12$	2·83	34·0	20 × 12	4·09	49·0
$9\frac{1}{2} \times 9\frac{1}{2}$	2·19	26·4	$13\frac{1}{2} \times 12$	2·91	34·9	22 × 12	4·27	51·3
10 × 10	2·32	27·9	14 × 12	3·07	36·9	24 × 12	4·12	49·5
$10\frac{1}{4} \times 10\frac{1}{4}$	2·45	29·4	15 × 12	3·14	37·8	26 × 12	3·96	47·5
$10\frac{1}{2} \times 10\frac{1}{2}$	2·49	29·9	16 × 12	3·30	39·6	30 × 12	3·35	40·2
11 × 11	2·51	30·3	17 × 12	3·35	40·1	...	...	...

The above table was calculated as follows:—

Let C = depth of web in clear and T = web thickness.

Then, treating the web as a strut, the ratio (l/r) of length or height to least radius of gyration $= C\sqrt{12} \div T$.

Let P = safe pressure (tons per square inch) corresponding to the ascertained ratio (l/r), where P = one-fourth of the calculated destructive pressure (by Fidler's formula as on page 157).

Then "safe direct load on web" per lineal inch, as tabulated above = P × T.

BROAD FLANGE BEAMS IN COTTON MILL.

STEELWORK BY MESSRS. WALMSLEY & SONS, BOLTON.

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GIRDERS CARRYING BRICK WALLS.

(1) The weight of a brick wall in tons will be found by calculating the superficial area (square feet) of the brickwork and multiplying by 4 for a 9″ wall, 6 for a 13½″ wall, 8 for an 18″ wall or 10 for a 22½″ wall. The result divided by 100 gives the weight of the wall in tons reckoned at 119½ lbs. per cubic foot.

(2) When calculating the weight of a wall, it is needless to say that allowance must also be made for the weight of roof and floors supported thereon.

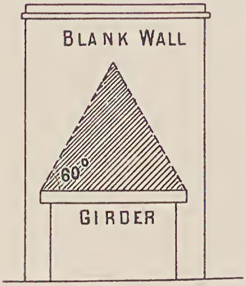


FIG. 1.

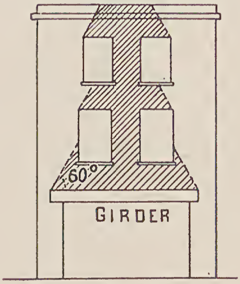


FIG. 2.

(3) In the case of a girder carrying a brick wall over an opening as in Fig. 1, it is usual in this country to proportion the girder to a distributed load equal to the weight of brickwork enclosed in the equilateral triangle shown in the sketch. This can be calculated by the following simple rule :—

Square the span (feet) and multiply by twice the thickness of the wall (inches). The result divided by 1,000 gives the uniformly distributed load (tons) to which the girder should be proportioned.*

In order to ensure perfect stiffness, a girder or girders should be selected which will carry the ascertained load with a deflection not exceeding 1/500th of the span, as in the table on page 122.

The load is comparatively insignificant as a rule and the size of beam is determined mainly by the necessity of providing a proper bearing surface, and of avoiding any appreciable deflection.

(4) If there are windows or other openings as in Fig. 2, the girder should be proportioned to a distributed load equal to the weight of brickwork enclosed in dotted lines.

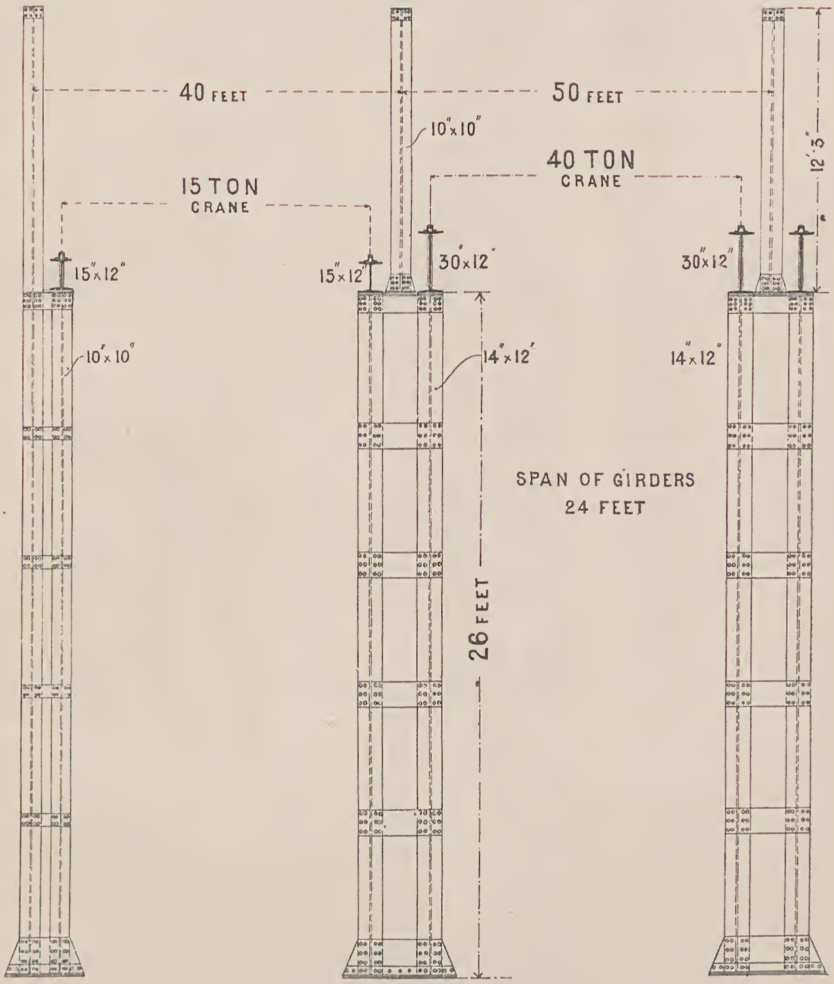
(5) When girders are used to support the whole or nearly the whole length of a wall, it is prudent to assume a distributed load equal to the entire weight of the wall, including, of course, the weight of floors and roof supported on the latter.

---

* (1) This rule is equivalent to taking the weight of brickwork as 124·2 lbs. per cubic foot.

(2) The more usual practice in the United States is to proportion the girder to a distributed load equal to the weight of brickwork enclosed in a triangle of which the height is two-thirds of the span.

CRANE-BEARING GIRDERS AND STANCHIONS IN FOUNDRY.



N.B.—The arrangement shown in the above drawing is similar to one adopted by Messrs. Robert Stephenson & Co., Ltd., Locomotive, Dock and Foundry Engineers, in a new foundry at their Newcastle Works (Contractors: Messrs. John Abbot & Co., Ltd., Newcastle-on-Tyne). The sections actually used by Messrs. Stephenson & Co. were B.F. Beams 30″ × 12″ and 20″ × 12″ for the girders (span: 24 feet average), 14″ × 12″ and 10″ × 10″ for the stanchions. The method of bracing and other minor details of construction differed from the above. The spans and loads were the same. The excellence and economy of the construction are obvious.

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Photo

ECONOMIC VALUES OF STANDARD ROLLED STEEL BEAMS.

BRITISH STANDARD BEAMS.					BROAD FLANGE BEAMS.				
Size in inches. Height. | Width.	Weight per foot. Lbs.	Section Modulus.	Economic Value.	Relative Value.	Nominal Size in inches. Height. | Width.	Weight per foot. Lbs.	Section Modulus.	Economic Value.	Relative Value.
..	..	..	..	..	30 × 12	177	492	2·78	..
..	..	..	..	..	26 × 12	166	408	2·46	..
24 × 7½	100	221	2·21	100	24 × 12	158	365	2·31	104·5
24 × 7½	100	221	2·21	100	..	..	..	..	..
..	..	..	..	..	22 × 12	152	324	2·13	..
20 × 7½	89	167	1·88	100	20 × 12	138	272	1·97	105·1
20 × 7½	89	167	1·88	100	..	..	..	..	..
..	..	..	..	..	19 × 12	128	244	1·91	..
18 × 7	75	128	1·71	100	18 × 12	121	219	1·81	106·0
18 × 7	75	128	1·71	100	..	..	..	..	..
..	..	..	..	..	17 × 12	113	196	1·74	..
16 × 6	62	90·7	1·46	100	16 × 12	107	177	1·65	113·1
15 × 6	59	83·9	1·42	100	15 × 12	101	159	1·57	110·8
15 × 5	42	57·1	1·36	100	15 × 12	101	159	1·57	115·8
14 × 6	46	62·9	1·37	100	14 × 12	96	144	1·50	109·7
..	..	..	..	..	13½ × 12	88	127	1·44	..
12 × 6	54	62·6	1·16	100	12 × 12	80	103	1·29	111·1
12 × 6	44	52·6	1·20	100	12 × 12	80	103	1·29	107·7
12 × 5	32	36·7	1·15	100	12 × 12	80	103	1·29	112·3
10 × 6	42	42·3	1·01	100	10 × 10	55	58·9	1·07	106·3
10 × 5	30	29·1	·970	100	10 × 10	55	58·9	1·07	110·4
10 × 5	30	29·1	·970	100	..	..	..	..	..
9 × 4	21	18·0	·857	100	8½ × 8½	44	40·9	·929	108·4
8 × 4	18	13·9	·772	100	8 × 8	37	31·6	·854	110·6
7 × 4	16	11·2	·700	100	7 × 7	31½	23·8	·756	107·9

NOTES TO THE ABOVE TABLE.

(1) The figures in the columns headed "Section Modulus" represent the carrying power of the various beams.

(2) The figures in the columns headed "Economic Value" represent the Section Modulus ÷ Weight per foot.

(3) The increased figures given in the columns headed "Relative Value" show by what percentage the economic value of each beam is greater or less than that of the British Standard Beam with which it is compared.

(4) The average relative values of the various sections tabulated, are as follows:—

British Standard Beams	100·0
Broad Flange Beams	109·3
American Standard Beams	94·4
Metric Standard Beams	96·4

ECONOMIC VALUES OF STANDARD ROLLED STEEL BEAMS.—Continued.

AMERICAN STANDARD BEAMS.					METRIC STANDARD BEAMS.				
Size in inches. Height. | Width.	Weight per foot. Lbs.	Section Modulus.	Economic Value.	Relative Value.	Size in inches. Height. | Width.	Weight per foot. Lbs.	Section Modulus.	Economic Value.	Relative Value.
..	..	..	..	..	..	..	..	..	..
..	..	..	..	..	..	..	..	..	..
24×7·25	100	198	1·98	89·6	..	..	..	..	..
24×7·00	80	174	2·18	98·4	..	..	..	..	..
..	..	..	..	..	..	..	..	..	..
20×7·14	90	156	1·73	92·4	19·69×7·28	94·1	168	1·79	95·2
20×6·25	65	117	1·80	95·9	..	..	..	..	..
..	..	..	..	..	..	..	..	..	..
18×6·26	70	102	1·46	85·4	17·72×6·69	77·3	125	1·62	94·8
18×6·00	55	88·4	1·61	94·2	..	..	..	..	..
..	..	..	..	..	..	..	..	..	..
..	..	..	..	..	15·75×6·10	61·7	89·0	1·44	98·6
15×6·00	60	81·2	1·35	95·2	14·96×5·87	56·0	77·0	1·38	96·7
15×5·50	42	58·9	1·40	103·1	14·96×5·87	56·0	77·0	1·38	101·1
..	..	..	..	..	14·17×5·63	50·9	66·4	1·31	95·4
..	..	..	..	..	..	..	..	..	..
12×5·61	55	53·5	·973	83·9	11·81×4·92	36·1	39·8	1·10	95·1
12×5·37	45	47·6	1·06	88·5	11·81×4·92	36·1	39·8	1·10	92·2
12×5·00	31½	36·0	1·14	99·6	11·81×4·92	36·1	39·8	1·10	96·1
10×5·10	40	31·7	·793	78·7	10·24×4·45	28·0	26·9	·961	95·4
10×4·81	30	26·8	·893	92·1	9·84×4·33	26·0	24·2	·931	95·9
10×4·66	25	24·4	·976	100·6	..	..	..	..	..
9×4·33	21	18·9	·900	105·0	9·06×4·02	22·4	19·2	·857	100
8×4·00	18	14·2	·789	102·2	7·87×3·54	17·5	13·1	·749	97·0
7×3·66	15	10·4	·693	99·0	7·09×3·23	14·6	9·82	·673	96·1

NOTES TO THE ABOVE TABLE.—Continued.

(5) Most of the American and Metric Standard sections have relative values under 100 showing that, on the whole, they are not so well designed as the British Standard sections. All of the Broad Flange Beam sections have relative values over 100, showing that they are the best designed and most economical beams of all.

(6) The relative values can be used to ascertain exactly what saving in weight would result from substituting Broad Flange Beams for an arrangement of ordinary joists of the same depth. For example, the relative value of a 16″×12″ beam as compared with a 16″×6″ joist is 113·1. This means that if 100 tons of 16″×12″ B.F. Beams will carry a given floor load, the required weight of 16″×6″ joists to carry the same load over the same area and span would be 113·1 tons, *i.e.* 13·1 % extra weight without any gain in strength.

(7) The sizes and properties of the American Standard sections are those published by the Carnegie Steel Company (Handbook, 1903).

N.B.—The above table is further explained on pages 132 etc.

COMPARATIVE VALUES OF DIFFERENT TYPES OF GIRDERS.

BROAD FLANGE BEAMS AND ORDINARY ROLLED STEEL JOISTS.

The carrying power of a rolled steel beam or compound girder is in exact proportion to its greatest Section Modulus.* That is to say, if one beam has a Section Modulus of 200 and another a Section Modulus of 100, the latter is exactly twice as strong as the former. This rule is independent of the shape or depth of the sections compared, and therefore constitutes a convenient basis of comparison.

The most economical type of beam is obviously that which has the highest carrying power in proportion to its weight, *i.e.* the highest Section Modulus in proportion to its weight per foot. This latter ratio, viz. Section Modulus ÷ weight per foot, may therefore be designated the "economic value" of a section. On page 130 a table is given showing the "economic values" of a number of standard H sections.

The practical utility of such comparisons is as follows:—

It is well known that the deeper the girder the greater the economy in weight. Therefore, in selecting floor beams, or in making any similar arrangement of girders, it is usual to decide firstly the maximum depth of beams which the circumstances permit; then, having specified the depth, it is desired to ascertain what is the lightest arrangement of girders that will carry the given floor load in the given depth.

The table on page 130 gives a direct answer to this question. It shows which sections are the most economical and to what extent.

If, for example, the load and span of a floor are such that 16″ beams are considered suitable, then the choice is between the various standard sections of this depth. Now the table shows the following "economic values":—

16″ B.F. Beams	...	...	1·65
16″ B.S. Joists	...	...	1·46
15¾″ Metric Joists	...	...	1·44

It shows therefore that Broad Flange Beams are the most economical and would make the lightest arrangement. It also points out that:—

$$1{\cdot}65 \quad : \quad 1{\cdot}46 \quad :: \quad 113{\cdot}1 \quad : \quad 100$$

In other words, it shows that the use of 16″ joists in place of 16″ Broad Flange Beams would entail 13·1 % extra weight of girders without any gain in strength.

Whatever depth of girders be taken, the table shows that the required weight of beams to carry a given floor load in a given depth, is reduced to a minimum by the use of wide-flanged girders.

The foregoing comparison between narrow-flanged joists and Broad Flange Beams is not quite complete, in so far as the comparative values or strengths of two different types of girders cannot be decided by merely considering the respective vertical loads which they will carry. Lateral rigidity in girders, when combined with adequate flange area for making connections of proportionate value, is an important element of strength. Consequently, any

* For definition of the term "Section Modulus" see page 8. It should be needless to say that this rule only applies to normal spans and loads as in floor beams, for example. It does not apply to exceptionally short beams liable to fail by vertical shearing or to exceptionally long beams, liable to side deflection. These points are dealt with in subsequent notes at the end of this section.

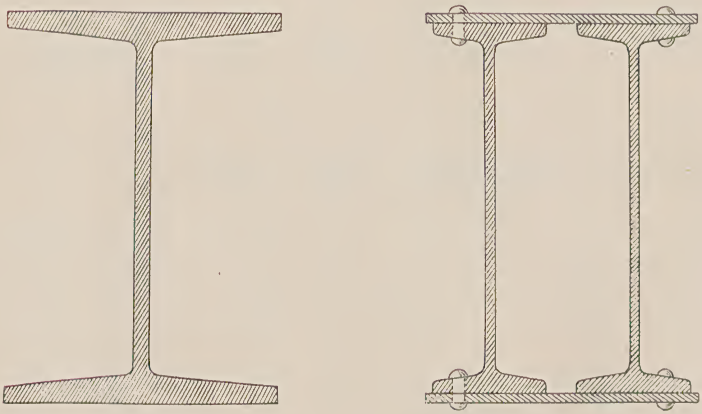
combination of wide-flanged girders must be deemed of greater strength than a similar combination of narrow-flanged girders designed to carry the same vertical loads.

This remark applies particularly to girders used in such structures as coal gantries, overhead railways, also crane-bearing girders in factories, dockyards etc. As compared with an ordinary railway bridge, the dead weight and consequent inertia of such structures is often comparatively small in proportion to the live load. Consequently, the upsetting tendency of the moving load and the many possibilities of cross strains are always carefully considered by those responsible for the design of such structures. The stability of the structure may depend on the resistance which members proportioned as girders may be able to offer to lateral flexure or compression, in which case deep narrow-flanged girders would be quite unsuitable.

The same remark applies to genuine steel-framed buildings and, in a lesser degree, to the steelwork in all buildings where girders are connected to stanchions. The latter are almost invariably subjected to eccentric loads and the consequent bending tendency must be provided against by due consideration of the lateral stability afforded by the end-fixing of the girders etc. [See also notes on "Connections," pages 12 etc.]

BROAD FLANGE BEAMS AND RIVETED GIRDERS.

NEW STYLE. OLD STYLE.



Fewer Joints, Greater Carrying Power, Less Weight, Less Cost.

See Table etc. overleaf and notes on page 136.

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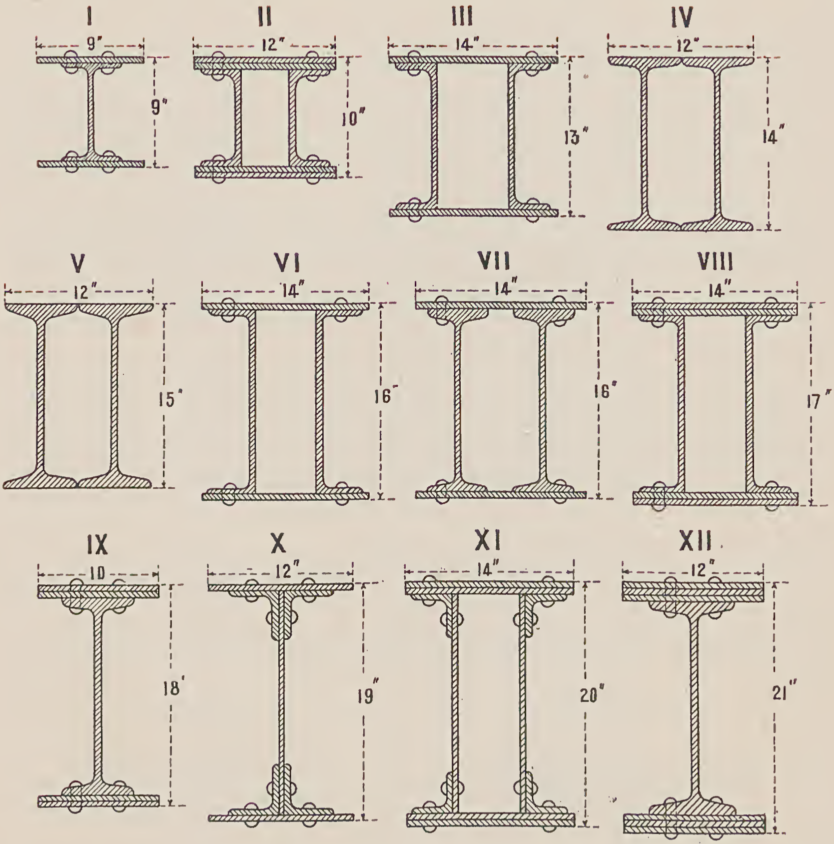
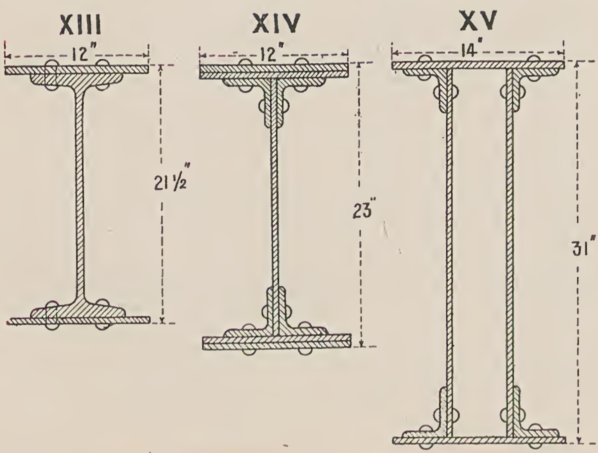
TYPICAL EXAMPLES OF RIVETED GIRDERS.





Particulars of these Riveted Girders are tabulated on the opposite page, where they are compared with Broad Flange Beams of corresponding strength.

TYPICAL EXAMPLES OF RIVETED GIRDERS COMPARED WITH BROAD FLANGE BEAMS.

Fig. No.	Size. Height.	Size. Width.	Weight per foot.	Materials used in making Riveted Girders.	Section Modulus.	Economic Value.	Relative Weight.	Relative Strength.	Safe Distributed Load. Span.	Safe Distributed Load. Load.
	Inches.		Lbs.		Ins.				Feet.	
Fig. I.	9	× 9	61	8″ × 5″ Joist ½″ plates	50·4	·826	120	96·5	14	18·0 tons
	9½	× 9½	51	Broad Flange Beam	52·2	1·02	100	100	14	18·6 tons
Fig. II.	10	× 12	130	8″ × 3½″ Channels ½″ plates	115	·885	153	100	16	35·9 tons
	12½	× 12	85	Broad Flange Beam.	115	1·35	100	100	16	35·9 tons
Fig. III.	13	× 14	115	12″ × 3½″ Channels ½″ plates	134	1·17	120	93·1	20	33·5 tons
	14	× 12	96	Broad Flange Beam	144	1·50	100	100	20	36·0 tons
Fig. IV.	14	× 12	92	14″ × 6″ Joists ..	126	1·37	105	99·2	22	28·6 tons
	13½	× 12	88	Broad Flange Beam	127	1·44	100	100	22	28·9 tons
Fig. V.	15	× 12	118	15″ × 6″ Joists ..	168	1·42	110	94·9	24	35·0 tons
	16	× 12	107	Broad Flange Beam	177	1·65	100	100	24	36·9 tons
Fig. VI.	16	× 14	134	15″ × 4″ Channels ½″ plates	188	1·40	119	95·9	26	36·2 tons
	17	× 12	113	Broad Flange Beam	196	1·74	100	100	26	37·7 tons
Fig. VII.	16	× 14	134	15″ × 5″ Joists ½″ plates	200	1·49	111	91·3	26	38·5 tons
	18	× 12	121	Broad Flange Beam	219	1·81	100	100	26	42·1 tons
Fig. VIII.	17	× 14	181	15″ × 4″ Channels ½″ plates	283	1·56	119	87·4	28	50·5 tons
	22	× 12	152	Broad Flange Beam	324	2·13	100	100	28	57·9 tons
Fig. IX.	18	× 10	132	16″ × 6″ Joist ½″ plates	221	1·67	103	90·6	30	36·8 tons
	19	× 12	128	Broad Flange Beam	244	1·91	100	100	30	40·7 tons
	18	× 12	121	Broad Flange Beam	219	1·81	.	. .	30	36·5 tons
Fig. X.	19	× 12	121	4″ × 4″ × ½″ Angles { Web ⅜″ plate; Flanges ½″ plates }	208	1·72	100	95·0	30	34·7 tons
	18	× 12	121	Broad Flange Beam	219	1·81	100	100	30	36·5 tons
Fig. XI.	20	× 14	209	3½″ × 3½″ × ½″ Angles ½″ plates	349	1·67	132	95·6	32	54·5 tons
	24	× 12	158	Broad Flange Beam	365	2·31	100	100	32	57·0 tons
Fig. XII.	21	× 12	200	18″ × 7″ Joist ½″ plates	406	2·03	121	99·5	34	59·7 tons
	26	× 12	166	Broad Flange Beam	408	2·46	100	100	34	60·0 tons
Fig. XIII.	21½	× 12	152	20″ × 7½″ Joist ¾″ plates	312	2·05	100	96·3	34	45·9 tons
	22	× 12	152	Broad Flange Beam	324	2·13	100	100	34	47·6 tons
Fig. XIV.	23	× 12	166	4″ × 4″ × ½″ Angles { Flanges ½″ plates; Web ⅜″ plate }	351	2·11	105	96·2	38	46·2 tons
	24	× 12	158	Broad Flange Beam	365	2·31	100	100	38	48·0 tons
Fig. XV.	31	× 14	202	3½″ × 3½″ × ½″ Angles ½″ plates	489	2·42	114	99·4	50	48·9 tons
	30	× 12	177	Broad Flange Beam	492	2·78	100	100	50	49·2 tons

NOTES TO THE ABOVE TABLE.

(1) The sizes and properties of the riveted girders illustrated on the opposite page are printed in black. The corresponding sizes and properties of Broad Flange Beams of approximately equal carrying power are printed in red. (The stated weights of Figs. IV. and V. do not include cast-iron separators; those of Figs. X. and XIV. do not include stiffeners.)

(2) The figures in the column headed "Economic Value" represent the Section Modulus ÷ weight per foot. They therefore show the ratio of carrying power to weight.

(3) The figures in the column headed "Relative Weight" show by what percentage, if any, the weight per foot of each riveted girder exceeds that of the B.F. Beam with which it is compared.

(4) The figures in the column headed "Relative Strength" show by what percentage, if any, the Section Modulus (*i.e.* the carrying power) of each riveted girder falls below the Section Modulus of the B.F. Beam with which it is compared.

(5) The table shows that ordinary compound or riveted girders can be replaced by B.F. Beams of approximately the same dimensions but of greater strength and of considerably less weight and cost.

(6) The average weight of the 15 riveted girders in the above table is 15½ % greater than that of the B.F. Beams, while the latter give an average gain in strength of nearly 5 %.

N.B.—The table is more fully explained on page 136.

COMPARATIVE VALUES OF DIFFERENT TYPES OF GIRDERS.—Continued.

BROAD FLANGE BEAMS AND RIVETED GIRDERS.

The table on page 135 enables comparison to be made between Riveted Girders and Broad Flange Beams. It shows that Broad Flange Beams can be substituted for fifteen representative examples of compound or riveted girders with an average saving in weight of 15½ % and an average gain in strength of nearly 5 % at the same time.

Though this result is extraordinary enough in itself, it does not do full justice to the remarkable economy and excellence of Broad Flange Beams as compared with riveted girders. In the first place, the cost of producing riveted girders is necessarily much greater than that of rolled steel beams. That there should be a considerable difference in cost and price is not surprising when the amount of workmanship required to make a riveted girder is fully realised. For example, to produce a single plate girder of the type illustrated in Fig. XIV., there are 9 different pieces to be cut to the same length; there are also some 729 holes to be marked out and drilled or punched, and about 243* rivets to close (4″ pitch) for every ton of this section turned out, to say nothing of the stiffeners forged to shape and riveted to flange and web at intervals throughout the length which are usual to this type of girder. Even in the simple compound in Fig. I. there are about 588 holes to be made and about 294 rivets to close (6″ pitch) for every ton produced. The saving in weight of 15½ %, considerable though it is, is therefore only a portion of the total economy. It should also be observed that the table on page 135 assumes that exactly the same working stress may be adopted in calculating the safe loads on riveted girders as for plain rolled steel beams. But, it is more usual and obviously reasonable, to employ a reduced working stress in calculating safe loads on riveted girders.† If 10 % is considered a reasonable allowance for the additional element of uncertainty in a riveted girder, the table on page 135 must be regarded as showing a saving in weight of 15½ % combined with a gain in strength of not 5 %, but 15 %.

COMPARISON WITH RIVETED GIRDERS OF EQUAL DEPTH.

The sizes of Riveted Girders and Broad Flange Beams chosen for comparison in the table on page 135 are those having as nearly as possible equal section moduli (*i.e.* nominally equal carrying power). It will be observed that most of the sizes compared are also very similar in outside dimensions. Comparison between dissimilar shapes has been avoided as far as possible, as it is reasonable to suppose that deep, narrow-flanged girders would not be used for the same purposes as wide and shallow girders. For some purposes, of course, useful comparison can only be made between girders of exactly the same depth. Consequently, in those examples where there is an appreciable difference in depth between the sections compared, the reader may compare the "economic values" of the riveted girders as tabulated on page 135 with the "economic values" of rolled steel beams of equal depth as tabulated on page 130. It will be found that, in every case, a substantial saving in weight is effected by the use of Broad Flange Beams.

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* The weight of these 243 rivets will be as nearly as possible 1 cwt., *i.e.* 5 % of the weight of the finished girder.

† Professor Adams recommends allowing 10 % reduction for steel "compound girders" as compared with "rolled steel joists" ("Designing Ironwork," Second Series, Part IV., page 12). The Carnegie Steel Co. allow 16,000 lbs. per square inch for "all rolled sections," and 15,000 lbs. per square inch for "Riveted beam box and plate girders" [*i.e.* 6¼ % reduction]. (Carnegie Handbook. Preface to Revised Edition of 1903.)

Supplementary Notes to the Table on page 135 ("Typical Examples of Riveted Girders").

(1) The meaning of the terms "Relative Weight" and "Relative Strength" will be obvious, if not already so, from the following example:—Taking the first example (Riveted Girder 9″×9″), the "Relative Weight" is stated to be 120, meaning that this girder is 20 % heavier than the 9½″×9½″ Beam with which it is compared. Similarly, the tabulated "Section Moduli" show the latter to be stronger than the riveted girder in the proportion of 52·2 to 50·4. The tabulated "Relative Strengths" show that 52·2 : 50·4 : : 100 : 96·5. That is, the carrying power of the riveted girder is 3½ % less than that of the 9½″ beam.

(2) In order to avoid the possibility of bias in specially designing typical riveted girders, the examples presented have been selected from the catalogue of a well-known firm of constructional engineers and girder merchants. The usual deductions have been made for rivet holes in calculating the section moduli. The weights per foot represent the approximate weights per foot of the finished girders, allowing of course for the rivet heads. These properties have been stated exactly as they appear in the catalogue referred to, a few of the examples having been verified and found correct.

DEFLECTION.

The deflection in inches at the centre of a beam supported at both ends and uniformly loaded is

$$D = \frac{5}{384} \times \frac{W.l^3}{E.I.} \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (1)$$

In the above formula: W = total load, E = modulus of elasticity of the steel, I = Moment of Inertia, l = span of beam in inches. The formula can also be written

$$D = \frac{5}{24.E.} \times \frac{l^2}{h} \times S \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (2)$$

where S = extreme fibre stress, h = depth of beam (inches). Taking E as 12,800 tons per square inch and S as $7\frac{1}{2}$ tons per square inch, this formula becomes:—

$$D = 0{\cdot}01758 \times \text{square of span (feet)} \div \text{depth of beam (inches)} \quad (3)$$

N.B.—Formulæ 2 and 3 only apply to symmetrical sections such as H beams; in the case of unsymmetrical sections, it is necessary to write $2C$ in place of h, where C = distance of centre of gravity of section from the extreme fibres.

The following instructive facts are evident from the above formulæ:—

(1) For any given working stress ($7\frac{1}{2}$ tons per square inch, say) all symmetrical steel beams of the same depth and span will show the same deflection when subjected to their full safe loads; the deflections of all such beams being in exact proportion to their depths, irrespective of the weights or strengths of the beams.

(2) The common belief that a "stiffer" girder is obtained by specifying steel of relatively high tensile strength, is shown by the above formula to be fallacious. The deflection depends simply on the elasticity of the steel (E) and this is practically constant for all grades of steel. In other words, both hard and soft steels exhibit a practically uniform change of length under the same stress up to their respective elastic limits. It follows that, as girders in actual practice are never loaded up to the elastic limit, the deflection of a steel girder has nothing to do with the hardness, carbon percentage or ultimate strength of the steel employed.

RATIO OF DEFLECTION TO SPAN.

The extent to which deflection in a beam is permissible is measured almost invariably by the ratio of deflection to span. In other words, if a deflection of $\frac{1}{2}''$ at the centre of a 15 feet span is unobjectionable, a deflection of 1″ at the centre of a 30 feet span is equally unobjectionable.

In some exceptional cases, obvious enough when they occur, it is the actual amount of deflection which is important. In such cases, the deflection, if not tabulated, must be calculated by the formula and, if found excessive, a deeper section must be used, or the tabulated load suitably reduced.

There are no theoretical objections (except in the case of loads rapidly fluctuating or applied with impact) to a degree of deflection greatly in excess of what would be practicable in building construction. Consequently, the permissible ratio of deflection to span varies according to circumstances.

If the live load is small in proportion to the weight of the floor, a shallower section of girder may be used than would otherwise be necessary, as that part of the deflection of a floor beam which is due to the permanent load is not objectionable.

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Beams shallow in proportion to their width are considerably stiffened when their ends are built into a brick wall* or strongly attached to stanchions. In an extreme case (*i.e.* in a continuous girder) the actual deflection would be only one-fifth of the deflection as calculated on the assumption of free ends.

This fact will not usually justify increasing the loads or the limit of span indicated above, but should be taken advantage of by employing the best practicable methods of fixing the ends of girders.

In the case of wide-flanged floor beams, especially when embedded in concrete, a deflection of 1/300th of the span (1/25th of an inch per foot) under the full estimated load, calculated on the assumption of free ends and ignoring the carrying capacity of the concrete, may be regarded as a safe outside allowance.

This limit is indicated by the zig-zag line in the table of safe distributed loads on Broad Flange Beams, page 120.

In the case of narrow-flanged joists (especially sections over 16″ deep), the ordinary end-fixing is much less effective than in the case of Broad Flange Beams or compound girders of like proportions. Consequently, it is desirable to limit the calculated deflection of the former to 1/30th of an inch per foot (*i.e.* 1/360th of the span) as has been done in this book; even then, the actual deflection, in practice, will probably be greater than that of wider or shallower sections calculated to produce a deflection of 1/25th of an inch per foot.

RATIO OF DEPTH OF BEAM TO SPAN.

Formula No. 3 on page 137 can be written as follows:—

$$l/h = D/l \times 8193 \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \qquad (4)$$

This formula shows that a given ratio of *deflection* to span (D/l), implies a corresponding ratio of *depth of beam* to span (h/l).

By substituting 1/300, 1/360 and 1/500 respectively for D/l in the above formula, the following results are obtained for beams loaded (uniformly) to a working stress of $7\frac{1}{2}$ tons per square inch:—

For a span equal to about 16 times depth of beam, the deflection = 1/500th of span.

For a span equal to about 23 times depth of beam, the deflection = 1/360th of span.

For a span equal to about 27 times depth of beam, the deflection = 1/300th of span.

Instead therefore of stating the permissible deflection as a given ratio of deflection to span, the same result can be attained by limiting the span to a given multiple of the depth of the beam. This assumes, however, that the beam is fully loaded to a working stress of $7\frac{1}{2}$ tons per square inch and only applies to uniformly distributed loads.

For example, in the early days of British railway engineering, the depth of plate girders was usually taken as 1/20th of the span. At 5 tons working stress, this corresponds to a deflection of about 1/600th of the span.†

But the same rule, when applied to floor beams, corresponds to a deflection of about 1/400th to 1/300th of the span, according to the working stress adopted.

Again, assuming an allowable working stress of $7\frac{1}{2}$ tons per square inch and an allowable deflection of 1/300th of the span, the correct depth of beam for

* According to Professor Adams:—"When the ends of rolled joists are built into a wall for some distance they do not constitute 'fixed ends' in the sense of producing stresses similar to those in a continuous girder, although a certain amount of stiffening takes place. With deep joists only a trifling result can be produced, but if shallow joists are built in for a length of at least four times their depth they will carry 50 % more load than if free."

† They are now more usually made of a depth equal to about 1/12th of the span.

a distributed load would be 1/27th of the span as already mentioned; but, for a load concentrated at the centre, the correct depth of beam would be 1/34th of the span.

It is necessary therefore to remember that rules which limit the ratio of depth to span are only indirect methods of limiting the ratio of *deflection* to span. The same end can sometimes be attained more economically by reducing the working stress.

This remark applies to cases where the loads to be carried are relatively trifling in proportion to the span, as in girders carrying brick walls over openings.

The *modus operandi* is simple :—

Suppose, for example, it is proposed to use an 18″ × 12″ beam over a span of 48 feet. The tabular load and deflection are given on page 121 as 23 tons and 2·3″ respectively. The deflection is not to exceed 1/300th of the span (*i.e.* 0·4″ per foot); that is, 1·92″.

To reduce the deflection from 2·3″ to 1·92″, the tabular load must be reduced in the same proportion, giving 19·2 tons as the required safe load.

SAFE LOADS CALCULATED FOR A GIVEN RATIO OF DEFLECTION TO SPAN.

If we substitute $l/500$ for the deflection (D) in the ordinary formula for deflection (No. 1, page 137), we find that

$$W = 13{\cdot}66 \times \text{Moment of Inertia} \div \text{square of span (feet)}.$$

W here represents the load which will cause a deflection equal to 1/500th of the span.

The table on page 122 is calculated by this formula and will be found useful in all cases where the selection of a beam has to be governed primarily by the deflection, or where exceptional stiffness is desired.

The notes accompanying the table provide a more detailed explanation.

Floor Girders.
H
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Foundations.
Misc. Shapes.
Misc. Tables.
Prices etc.
Code.
Grey Mill.
Tests etc.
R.I.B.A.
Photos.

SHEARING STRESSES IN GIRDERS.

The average resistance to shear of iron and steel is about four-fifths of the tensile strength. In the case of rolled steel girders, the shear is generally assumed to be distributed over the area D T, where D=depth of beam and T=web thickness. On this assumption, the maximum shearing stress in a beam carrying a uniformly distributed load, W will be $\frac{1}{2}$W÷D T. [For other cases of loading, see page 124.] Usually, the web tends to buckle rather than to fail by direct shear, so that it is not sufficient to consider merely the shearing strength. Engineers in this country and in the United States now usually calculate the safe shearing stresses in beams by formulæ which represent the tendency of the web to fail laterally as a strut, when subject to compressive stresses acting at an angle of 45° to the vertical and equal in intensity to the vertical shearing stress.

The table below shows the maximum shearing stresses and corresponding safe distributed loads on Broad Flange Beams calculated in this manner.

It will be observed that, in the case of the smaller sizes of Broad Flange Beams, the safe shearing stresses calculated for buckling are approximately equal to the stress allowable for simple shear (say 5 tons per square inch); but the webs of the deepest sections, like ordinary joists, show a greater tendency to fail by buckling than by actual shear.

Calculating the safe shearing stresses for one or two typical joist sections, in the same manner as for Broad Flange Beams, we have the following results:—

					Tons per square inch.
For R.S. Joist,	12″ × 6″ × 44 lbs.,	safe shearing	stress	=	3·68
,, ,,	14″ × 6″ × 46 lbs.	,,	,,	=	2·86
,, ,,	16″ × 6″ × 62 lbs.	,,	,,	=	3·68
,, ,,	18″ × 7″ × 75 lbs.	,,	,,	=	3·15

Section	Safe Maximum Shearing Stress.	Maximum Safe Distributed Load.	Section.	Safe Maximum Shearing Stress.	Maximum Safe Distributed Load.
	Tons per sq. in.	Tons.		Tons per sq. in.	Tons.
7″ × 7″	(5)	(23·4)	14″ × 12″	4·45	71
8″ × 8″	4·81	25	15″ × 12″	4·29	74
$8\frac{1}{2}$″ × $8\frac{1}{2}$″	4·66	28	16″ × 12″	4·29	82
$9\frac{1}{2}$″ × $9\frac{1}{2}$″	4·74	35	17″ × 12″	4·11	87
10″ × 10″	4·77	38	18″ × 12″	4·15	99
$10\frac{1}{4}$″ × $10\frac{1}{4}$″	4·81	42	19″ × 12″	4·0	103
$10\frac{1}{2}$″ × $10\frac{1}{2}$″	4·74	44	20″ × 12″	4·19	125
11″ × 11″	4·7	47	22″ × 12″	4·04	142
$11\frac{1}{2}$″ × $11\frac{1}{2}$″	4·74	51	24″ × 12″	3·64	141
12″ × 12″	4·77	55	26″ × 12″	3·31	141
$12\frac{1}{2}$″ × 12″	4·61	59	30″ × 12″	2·57	126
$13\frac{1}{2}$″ × 12″	4·45	63			

N.B.—The above "Maximum Safe Loads" are given in Part I. under the various sizes and are also printed in italics in the table of safe distributed loads on page 120.

Let C = depth of web (inches) in clear between flanges.
Let l = ,, ,, ,, ,, measured at an angle of 45° to the flanges.
Then $l = C\sqrt{2}$ .. .. (1)
Let T = web thickness (inches).
Let r = radius of gyration of web as a strut.
Then $r = T \div \sqrt{12}$ .. .. .. (2)
and $l/r = C\sqrt{24} \div T$ .. .. .. (3)
Let P = one-fourth of the destructive pressure per square inch calculated by Fidler's formula (as on page 157) for a strut of the ascertained l/r.
Then P = Safe maximum shearing stress per square inch in web as tabulated above.
Let D = Depth of beam (inches) and T = web thickness as before.
Then W = Maximum safe distributed load as tabulated above = 2 P × D T.

SHEARING STRESSES IN GIRDERS.—Continued.

It will be observed that the safe shearing stresses for ordinary joists are lower than those for Broad Flange Beams. This might naturally be expected as the webs of ordinary joists are usually thinner in proportion to their depth than the webs of Broad Flange Beams.

The inferiority of ordinary joists from this point of view is greater than the above figures suggest. It is assumed, in fact, that the buckling tendency of the web is in each case similar to that of a strut with fixed ends. This is approximately true of wide-flanged sections, but in the case of deep sections with narrow flanges, the conditions more nearly resemble those of a strut with pivoted ends.

Another point which renders the above comparison unduly favourable to narrow-flanged sections is the well-known fact that the shear in an H beam is not limited to the section of the web as it is always assumed to be in calculations. Under ordinary circumstances a $12'' \times 12'' \times \frac{1}{2}''$ beam would unquestionably carry a greater load than a $12'' \times 6'' \times \frac{1}{2}''$ beam however short the span, although the formula attributes equal strength to both sections for spans under 9·4 feet.

LATERAL STRENGTH OF BEAMS.

The tables of safe distributed loads are calculated as usual on the assumption that the beams receive the necessary side support to prevent lateral deflection. This side support is usually found in ordinary building work in the shape of cross-joists etc., or the beams may be sufficiently secured against flexure in this direction by the flooring itself. If, however, no suitable provision exists, the beams should be supported laterally by tie-rods at intervals of about 20 times the flange width, through upper portion of web. The necessity of providing side support follows from consideration of the upper or compression flange of a beam as a strut. Beams may be used over spans exceeding 20 times the flange width without any lateral support, if the tabular loads are suitably reduced. The following rule for effecting such reduction is considered good practice: for a span of 25 times the flange width, reduce the tabular load by 5 %; for a span of 26 times, reduce by 6 %; for 27 times, by 7 %, and so on. It should be noted that the lateral stiffness of a solid steel beam is considerably enhanced by the continuity of its section as compared with many types of built-up girder. In the case of a lattice girder, for example, the upper flange receives but little support from the flange in tension.

Broad Flange Beams are obviously superior to ordinary R.S. Joists in this respect and may be employed over much longer spans without lateral support. This advantage of wide-flanged beams makes them particularly suitable for use as floor beams in brick or concrete arched floors, for crane girders etc.

N.B.—In calculating the special table of safe loads for a deflection not exceeding 1/500th of the span (page 122), it was necessary to reduce the working stress. The reductions thus made incidentally provide amply against lateral failure, except in the case of sections over 17 inches deep. In the case of sections 18 to 24 inches deep, the loads printed in large type may be applied in full; but some of the loads in small type should be reduced, according to the above rule, if the beam is without lateral support. In the case of sections 26 and 30 inches deep, most of the tabulated loads should be reduced if the beam is without side support.

TABLE FOR SELECTING FLOOR GIRDERS FOR VARIOUS LOADS AND SPANS.

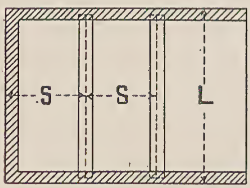
Four suitable depths of Broad Flange Beams are given for each span and the table states the correct spacing of these girders, centre to centre in feet, for various floor loads (cwts. per foot super). For suitable reinforcing joists, if required, see table overleaf.

Span of Beams.	Suitable Depths of Beams.				$1\frac{1}{2}$ cwts. per square foot.				$1\frac{3}{4}$ cwts. per square foot.				2 cwts. per square foot.			
	A	B	C	D	A	B	C	D	A	B	C	D	A	B	C	D
	Ins.	Ins.	Ins.	Ins.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.
16 ft.	...	7	8	10	...	6·2	8·2	15·3	...	5·3	7·0	13·1	...	4·6	6·2	11·5
17 ,,	7	8	$8\frac{1}{2}$	10	5·2	7·3	9·5	13·6	4·5	6·3	8·1	11·7	3·9	5·5	7·1	10·2
18 ,,	7	8	$9\frac{1}{2}$	11	4·4	6·5	10·7	17·1	3·8	5·6	9·2	14·7	3·3	4·9	8·1	12·8
19 ,,	8	$8\frac{1}{2}$	$9\frac{1}{2}$	12	5·5	7·6	9·6	19·0	4·7	6·5	8·3	16·3	4·1	5·7	7·2	14·3
20 ,,	8	$9\frac{1}{2}$	10	12	4·7	8·7	9·8	17·2	4·0	7·5	8·4	14·7	3·5	6·5	7·4	12·9
21 ,,	$8\frac{1}{2}$	$9\frac{1}{2}$	11	14	5·8	7·9	12·6	21·8	5·0	6·8	10·8	18·7	4·3	5·9	9·4	16·3
22 ,,	$9\frac{1}{2}$	10	11	14	7·0	8·1	11·5	19·8	6·0	6·9	9·8	17·0	5·3	6·1	8·6	14·9
23 ,,	$9\frac{1}{2}$	11	12	15	6·2	10·5	13·0	20·0	5·3	9·0	11·1	17·2	4·6	7·9	9·7	15·0
24 ,,	10	11	12	15	6·4	9·6	11·9	18·4	5·5	8·2	10·2	15·8	4·8	7·2	8·9	13·8
25 ,,	10	12	14	16	5·6	11·0	15·4	18·9	4·8	9·4	13·2	16·2	4·2	8·2	11·5	14·2
26 ,,	10	12	14	16	5·0	10·2	14·2	17·5	4·3	8·7	12·2	15·0	3·7	7·6	10·6	13·1
27 ,,	11	12	14	16	7·1	9·4	13·2	16·2	6·0	8·1	11·3	13·9	5·3	7·1	9·9	12·1
28 ,,	12	14	15	17	8·4	12·3	13·5	16·7	7·2	10·5	11·6	14·3	6·3	9·2	10·1	12·5
29 ,,	12	14	15	18	7·5	11·4	12·6	17·4	6·5	9·8	10·8	14·9	5·6	8·6	9·5	13·0
30 ,,	12	14	15	18	6·8	10·7	11·8	16·2	5·8	9·1	10·1	13·9	5·1	8·0	8·8	12·2
32 ,,	12	15	16	19	5·6	10·4	11·5	15·9	4·8	8·9	9·9	13·6	4·2	7·8	8·6	11·9
34 ,,	14	15	17	20	7·9	9·2	11·3	15·7	6·8	7·9	9·7	13·5	5·9	6·9	8·5	11·8
36 ,,	14	16	18	22	6·6	9·1	11·3	16·7	5·7	7·8	9·7	14·3	5·0	6·8	8·4	12·5
38 ,,	15	18	19	24	6·6	10·1	11·3	16·9	5·6	8·7	9·7	14·4	4·9	7·6	8·4	12·6
40 ,,	16	18	20	26	6·6	9·1	11·3	17·0	5·6	7·8	9·7	14·6	4·9	6·8	8·5	12·7
45 ,,	18	20	24	30	6·5	9·0	12·0	16·2	5·5	7·7	10·3	13·9	4·9	6·7	9·0	12·1

EXPLANATION OF THE ABOVE TABLE.

Suitable floor beams can be selected from this table for any type of floor, concrete or otherwise.

The accompanying sketch represents a skeleton plan of the girders in a floor, L representing the span and S the "spacing" of the girders. The table gives the correct values of S, *i.e.* the maximum safe spacing of the girders centre to centre in feet, for various spans and six different floor loads. Assuming the weight of the flooring to average $\frac{3}{4}$ cwt. per square foot, it is recommended that the total floor load be taken as :—



$1\frac{1}{4}$ to $1\frac{1}{2}$ cwts. per square foot for dwelling houses, hotels, hospitals, lodging houses etc.

$1\frac{3}{4}$ to 2 cwts. per square foot for office buildings, places of public assembly, small workshops, retail shops etc.

$2\frac{1}{2}$ to 3 cwts. per square foot for ordinary warehouse floors.

For an estimated floor load of $1\frac{1}{4}$ cwts. per square foot, take twice the spacing as given in the table for a load of $2\frac{1}{2}$ cwts. per square foot. Similarly, for any other assumed load not given in the table, vary the spacing in the same proportion as the load is varied.

The following example will suffice to explain the table :—

Suppose that the span of the girders is 36 feet; for this span, the table gives a choice of four Broad Flange Beams 14″, 16″, 18″ and 22″ deep respectively. These four sizes are headed A, B, C and D respectively. Now suppose that the floor load is estimated at 3 cwts. per square foot; then the table shows that any one of the four following arrangements would be suitable :—

Either 14 inch Broad Flange Beams spaced at 3·3 feet centres.
Or 16 ,, ,, ,, ,, at 4·5 feet ,,
Or 18 ,, ,, ,, ,, at 5·6 feet ,,
Or 22 ,, ,, ,, ,, at 8·3 feet ,,

It will be observed that the table states the nominal *depths* of the beams; the rest of the dimensions can be ascertained by referring to Part I.

TABLE FOR SELECTING FLOOR GIRDERS FOR VARIOUS LOADS AND SPANS.—Continued.

Span of Beams.	Suitable Depths of Beams.				$2\frac{1}{2}$ cwts. per square foot.				$2\frac{3}{4}$ cwts. per square foot.				3 cwts. per square foot.			
	A	B	C	D	A	B	C	D	A	B	C	D	A	B	C	D
	Ins.	Ins.	Ins.	Ins.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.
16 ft.	...	7	8	10	...	3·7	4·9	9·2	...	3·4	4·5	8·4	...	3·1	4·1	7·7
17 ,,	7	8	8½	10	3·1	4·4	5·7	8·2	2·8	4·0	5·2	7·4	2·6	3·6	4·7	6·8
18 ,,	7	8	9½	11	2·6	3·9	6·4	10·3	2·4	3·5	5·9	9·3	2·2	3·2	5·4	8·6
19 ,,	8	8½	9½	12	3·3	4·5	5·8	11·4	3·0	4·1	5·3	10·4	2·7	3·8	4·8	9·5
20 ,,	8	9½	10	12	2·8	5·2	5·9	10·3	2·6	4·7	5·4	9·4	2·3	4·3	4·9	8·6
21 ,,	8½	9½	11	14	3·5	4·7	7·5	13·1	3·2	4·3	6·9	11·9	2·9	3·9	6·3	10·9
22 ,,	9½	10	11	14	4·2	4·9	6·9	11·9	3·8	4·4	6·2	10·8	3·5	4·0	5·7	9·9
23 ,,	9½	11	12	15	3·7	6·3	7·8	12·0	3·4	5·7	7·1	10·9	3·1	5·2	6·5	10·0
24 ,,	10	11	12	15	3·8	5·8	7·1	11·0	3·5	5·2	6·5	10·0	3·2	4·8	6·0	9·2
25 ,,	10	12	14	16	3·4	6·6	9·2	11·3	3·1	6·0	8·4	10·3	2·8	5·5	7·7	9·4
26 ,,	10	12	14	16	3·0	6·1	8·5	10·5	2·7	5·5	7·7	9·5	2·5	5·1	7·1	8·7
27 ,,	11	12	14	16	4·2	5·6	7·9	9·7	3·9	5·1	7·2	8·8	3·5	4·7	6·6	8·1
28 ,,	12	14	15	17	5·0	7·3	8·1	10·0	4·6	6·7	7·4	9·1	4·2	6·1	6·8	8·3
29 ,,	12	14	15	18	4·5	6·9	7·6	10·4	4·1	6·2	6·9	9·5	3·8	5·7	6·3	8·7
30 ,,	12	14	15	18	4·1	6·4	7·1	9·7	3·7	5·8	6·4	8·8	3·4	5·3	5·9	8·1
32 ,,	12	15	16	19	3·4	6·2	6·9	9·5	3·1	5·6	6·3	8·7	2·8	5·2	5·8	7·9
34 ,,	14	15	17	20	4·7	5·5	6·8	9·4	4·3	5·0	6·2	8·6	3·9	4·6	5·7	7·8
36 ,,	14	16	18	22	4·0	5·5	6·8	10·0	3·6	5·0	6·1	9·1	3·3	4·5	5·6	8·3
38 ,,	15	18	19	24	3·9	6·1	6·8	10·1	3·6	5·5	6·1	9·2	3·3	5·1	5·6	8·4
40 ,,	16	18	20	26	3·9	5·5	6·8	10·2	3·6	5·0	6·2	9·3	3·3	4·6	5·7	8·5
45 ,,	18	20	24	30	3·9	5·4	7·2	9·7	3·5	4·9	6·6	8·8	3·2	4·5	6·0	8·1

EXPLANATION OF THE ABOVE TABLE.—CONTINUED.

Which of the four sizes of Beams to use is a question of practical convenience or economy.

The beams in columns B and C are of the depths most commonly adopted for floor beams.

The beams in column A are unusually shallow and are suitable for stationary loads, as in ordinary buildings, in cases where headroom is of importance. The deflection of these beams will not be excessive if they are spaced in accordance with the table.

The beams in column D are good economical sizes for use in cases where headroom is not particularly important.

Whichever sizes are used, the required weight of girders will be substantially less than the weight of any other possible combination of girders of the same depth and same factor of safety, whether comparison be made with narrow-flanged joists or compound girders. (See pages 132 and 136.)

The total weight of the steelwork in any given floor can be estimated very closely as follows:—Divide the weight per foot (lbs.) of the main girders by the spacing (feet). The result gives the average weight of girders in lbs. per foot super. Similarly, divide the weight per foot of the reinforcing joists, if used, by their spacing, and the result gives the average weight of these in lbs. per foot super.

N.B.—The spacings of the beams in columns B, C and D have been calculated for a working stress of $7\frac{1}{2}$ tons per square inch. Consequently, the estimated loads on these girders and corresponding (calculated) deflections will be identical with those given in the ordinary table of safe loads on page 120.

If the tabulated spacing is reduced, the working stress and deflection will be reduced in exactly the same proportion. *E.g.* If it is desired to reduce the working stress from $7\frac{1}{2}$ to 6 tons per square inch, the spacing must be reduced by one-fifth.

The spacings of the beams in column A have been calculated for a deflection of 1/300th of the span, assuming free ends. The formula is given on page 139. If the tabulated spacing is reduced, the deflection will be reduced in the same proportion. *E.g.* If it is desired to reduce the deflection from 1/300th to 1/400th of the span, the tabular spacing must be reduced by one-fourth. The working stress in these beams (column A) is less than $7\frac{1}{2}$ tons per square inch; it will be $17\, h \div L$, where h is the depth of the beam (inches) and L the span (feet).

TABLES FOR SELECTING SUITABLE REINFORCING JOISTS FOR B.F. BEAM TYPES OF CONCRETE FLOORING.

Spacing of Main Girders centre to centre.	Total Floor Load per square foot.					
	$1\frac{1}{2}$ cwts.	$1\frac{3}{4}$ cwts.	2 cwts.	$2\frac{1}{2}$ cwts.	$2\frac{3}{4}$ cwts.	3 cwts.
7 feet ...	·45	·60	·75	1·05	1·20	1·35
8 ,, ...	·61	·82	1·02	1·43	1·63	1·83
9 ,, ...	·80	1·07	1·34	1·86	2·13	2·40
10 ,, ...	1·01	1·35	1·69	2·36	2·70	3·04
11 ,, ...	1·25	1·67	2·09	2·91	3·33	3·75
12 ,, ...	1·50	2·02	2·52	3·53	4·03	4·53
13 ,, ...	1·80	2·40	3·00	4·20	4·80	5·40
14 ,, ...	2·11	2·82	3·52	4·93	5·63	6·33
15 ,, ...	2·45	3·27	4·09	5·71	6·53	7·35
16 ,, ...	2·81	3·75	4·69	6·56	7·50	8·44
17 ,, ...	3·20	4·27	5·34	7·46	8·53	9·60
18 ,, ...	3·61	4·82	6·02	8·43	9·63	10·8
19 ,, ...	4·05	5·40	6·75	9·45	10·8	12·2
20 ,, ...	4·51	6·02	7·52	10·5	12·0	13·5

EXPLANATION OF THE ABOVE TABLE.

(1) Before using the above table it is necessary to decide on the size and spacing of the main girders by means of the table on page 142 overleaf. Then the above table states what "Section Modulus" the reinforcing joists must have, according to the spacing of the main girders and total floor load. Having ascertained the required "Section Modulus," refer to the second table below for a suitable size of joist. The size of joist thus arrived at will be adequate if spaced at $2\frac{1}{2}$ feet centres or closer.

(2) If the pitch is increased, increase the tabulated "Section Modulus" in the same proportion. *Suppose that the main girders are spaced 10 feet apart, the total floor load being $2\frac{3}{4}$ cwts. per square foot. The above table gives the required "Section Modulus" as 2·70. The table below shows that $4\frac{3}{4}''$ joists have the required section modulus. But, if the joists are spaced at 4 feet centres, instead of $2\frac{1}{2}$ feet, the required section modulus will be* $\frac{4}{2\frac{1}{2}} \times 2{\cdot}70 = 4{\cdot}32$.

(3) The above procedure enables a joist to be chosen to suit a given pitch. It may, however, be desired to ascertain the maximum safe pitch for a given size of joist. Reverting to the above example; if it is decided to use $4\frac{3}{4}''$ joists, the section modulus of which is given below as 2·85, the maximum safe spacing will obviously be $\frac{2{\cdot}85}{2{\cdot}70} \times 2\frac{1}{2} = 2{\cdot}64$ feet centre to centre.

N.B.—This table has been calculated with special reference to the Broad Flange Beam types of concrete floor as illustrated and explained in the following pages. It must not be applied to other types of floor without due allowance for the basis of calculation.

BASIS OF CALCULATION.

Let S = spacing of main girders centre to centre (feet).
Let Z = superimposed or "live" load on floor (cwts. per square foot) = stated floor load less $\frac{3}{4}$ cwt.
Let M = tabulated section modulus.

Then the *clear* span of each joist, the main girders being Broad Flange Beams, will be approximately S − 1 feet. If pitched at $2\frac{1}{2}$ feet centres, the proportion of the live load on each joist will be $2\frac{1}{2}$ (S − 1) Z cwts. = X tons, say. Then M represents the required section modulus of a beam, with fixed ends, carrying a distributed load of X tons on a span of S − 1 feet, at $7\frac{1}{2}$ tons per square inch working stress. It will be observed that the ends of the reinforcing joists are simply assumed to be fixed in the block of concrete held between the wide flanges of the main girders, and that the concrete flooring is treated as carrying its own weight. This point is dealt with further on page 151.

LIST OF SUITABLE REINFORCING JOISTS, WITH SECTION MODULI.

BRITISH STANDARD JOISTS.			METRIC STANDARD JOISTS.		
Reference No.	Size.	Section Modulus.	Reference No.	Size (depth and weight).	Section Modulus.
203	$3'' \times 1\frac{1}{2}'' \times 4$ lbs.	1·11	310	3·15″ × 4·0 lbs.	1·18
209	$4'' \times 1\frac{3}{4}'' \times 5$ lbs.	1·84	313	3·54″ × 4·7 lbs.	1·58
206	$3'' \times 3'' \times 8\frac{1}{2}$ lbs.	2·53	316	3·94″ × 5·6 lbs.	2·08
215	$4\frac{3}{4}'' \times 1\frac{3}{4}'' \times 6\frac{1}{2}$ lbs.	2·85	319	4·33″ × 6·4 lbs.	2·64
212	$4'' \times 3'' \times 9\frac{1}{2}$ lbs.	3·76	322	4·72″ × 7·5 lbs.	3·33
218	$5'' \times 3'' \times 11$ lbs.	5·45	325	5·12″ × 8·5 lbs.	4·09
224	$6'' \times 3'' \times 12$ lbs.	6·74	328	5·51″ × 9·5 lbs.	4·99
233	$7'' \times 4'' \times 16$ lbs.	11·2	331	5·91″ × 10·7 lbs.	5·97
236	$8'' \times 4'' \times 18$ lbs.	13·9	340	7·09″ × 14·6 lbs.	9·82

BROAD FLANGE BEAMS AS FLOOR GIRDERS.

CONCRETE FLOORS.

The advantages of Broad Flange Beams are not restricted to any particular types of flooring nor to any particular methods of reinforcing concrete.

When concrete or timber flooring is carried on steel girders, Broad Flange Beams are by far the most suitable and economical* type of girders to employ. The wide flanges provide ample accommodation for the landing or housing of timber or steel joists. As regards arched floors of brick or concrete, the lateral resistance of wide-flanged sections makes them particularly suitable, and the inside width of the flanges provides an excellent abutment for the arches.

For flat concrete floors of no matter what type, Broad Flange Beams offer an exceptional advantage on account of the relatively large surface of the beam available for the adhesion of the concrete. This is a feature of considerable importance, because steel entering into the construction of a floor through most of its depth, cuts up the concrete and partially destroys the continuity of the construction.

It is obviously desirable to preserve the bond of a floor by any means which does not add to the cost. Depth for depth, Broad Flange Beams offer over 25 % more adhesion surface than ordinary rolled steel joists or compound girders and twice the depth of "hold" in the flanges. Furthermore, it is evident that the wide and shallow block of concrete confined between the flanges of Broad Flange Beams is of very great value for the end-fixing of any kind of reinforcing metal. When the ends of light joists are embedded in this concrete, the joists have a carrying value approaching that of fixed ends, which increases their normal carrying power by 50 %. The further advantages of Broad Flange Beams over all other types of steel girders in reducing weight and cost are fully explained on pages 132 to 136.

FIRE PROTECTION OF WIDE-FLANGED GIRDERS.

Broad Flange Beams might, at first sight, appear at a disadvantage as compared with ordinary joists in having a wider flange to protect from exposure to fire.† But, as one wide-flanged beam will do the work of two narrow-flanged beams, what is really involved is the question of empanelling *one* 12″ flange, say, instead of two 6″ flanges, which is certainly neither more troublesome nor more expensive. If, on the other hand, the comparison is between one wide-flanged girder and one narrow-flanged girder of the same strength, the advantage is unquestionably on the side of the wide-flanged girder, because the depth of the encasement below the level of the ceiling will be reduced by about 4 inches, and the exposed surface of the encasement by at least 10 %. For suitable form of encasements to Broad Flange Beams, see notes on page 151, and Fig. 6 on page 148.

"B.F. BEAM TYPES" OF CONCRETE FLOORING.

The following drawings illustrate some exceptionally simple and economical types of concrete flooring, which are rendered practicable by the use of wide-flanged girders. The first three isometrical drawings, Figs. 1, 2 and 3, are simply modifications of the same common principle, that

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* For comparison with narrow-flanged joists, see page 132 and with riveted girders page 136.

† On this point, the attention of readers is particularly invited to the result of the test conducted by the British Fire Prevention Committee on the 24th February, 1906 (see summary on page 152).

B.F. BEAM TYPES OF CONCRETE FLOORING.

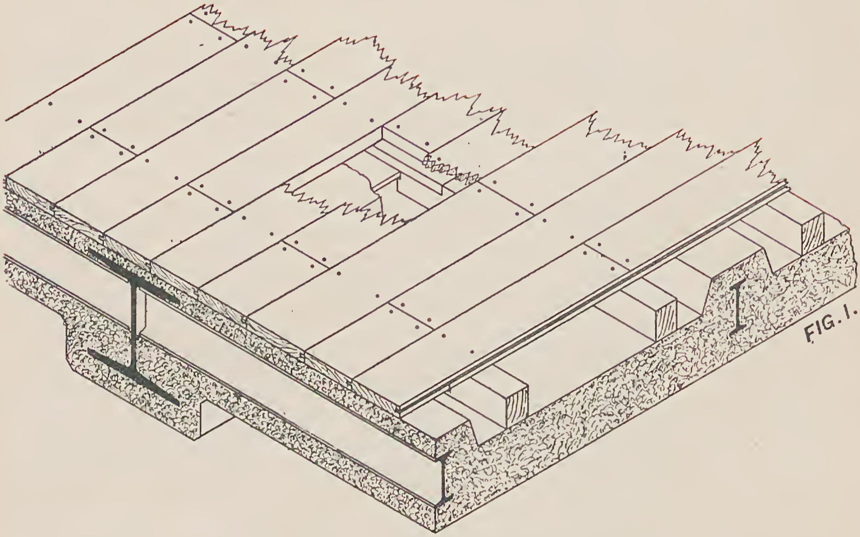


FIG. 1.

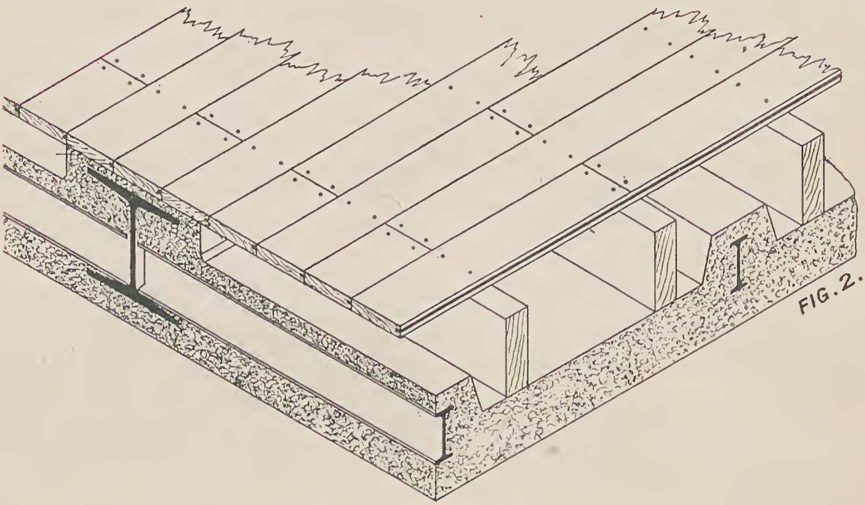


FIG. 2.

B.F. BEAM TYPES OF CONCRETE FLOORING

Continued.

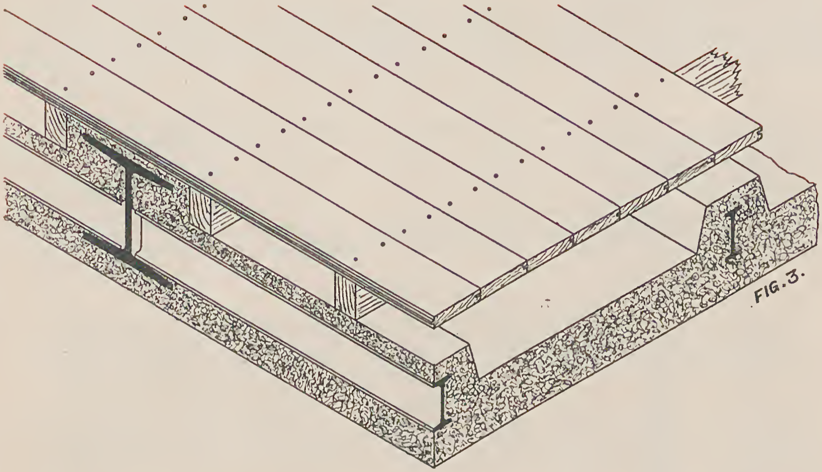


FIG. 3.

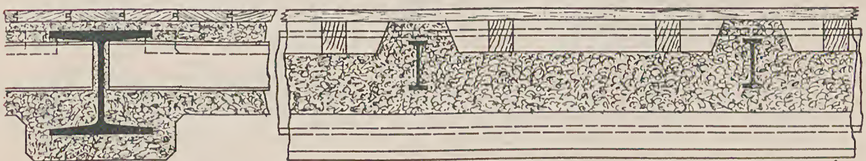


FIG. 1A.

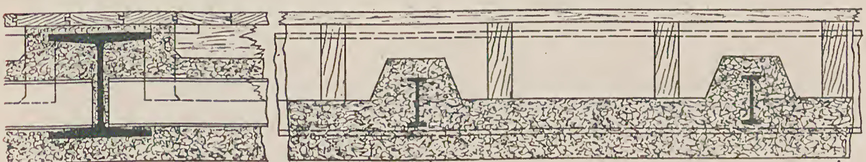
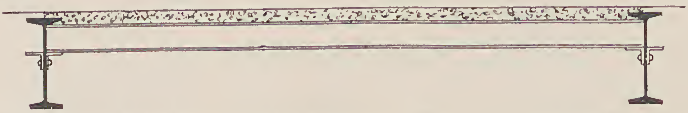


FIG. 2A.

FIG. 3A.

girders. The weight of the reinforcing joists will not usually exceed $2\frac{1}{2}$ lbs. per square foot, say $3\frac{1}{2}$ % of the weight of the concrete. Nearly the same percentage of metal of a more expensive character (twisted bars etc.) is used in some of the best patented types of reinforced concrete.

(*c*) When using ordinary joists as the main girders, angles have to be riveted to the webs as in the accompanying sketch, for the deep and narrow



block of concrete held between the flanges is obviously inadequate even as a simple bearing surface for the ends of the joists. This "cleating" is rendered unnecessary when using Broad Flange Beams. In many cases, the requisite girders can be despatched to the site direct from the mills or from stock, which means a saving not only of the cost of the extra material and workmanship, but also of the cost of extra carriage and double handling, which is often a considerable item.

(*d*) A considerable saving in headroom may be effected, reducing the cost of brickwork etc.

(*e*) The whole of the materials required are of the commonest description such as the builder can procure from stock, when necessary, in all important localities.

(*f*) The proprietors of patented systems have to incur considerable risk and expense in advertising of all kinds. Moreover, for every contract secured, it is probable that they have been put to the expense of furnishing designs and estimates without success in twenty other cases. The cost of these unsuccessful attempts must be paid for out of the contracts obtained and, consequently, the margin of profit is necessarily a relatively high one. No part of the Broad Flange Beam types of floor illustrated in this book is patented, and competitive tenders may be invited from any number of respectable firms with the assurance that all of them will be able to obtain the necessary supplies promptly and at the lowest wholesale prices. This fact in itself will probably mean a saving in cost of 10 % at the least, quite apart from the actual cost of the work and materials.

DETAILS OF CONSTRUCTION.

MAIN GIRDERS.

The floor load is estimated at so many cwts. per foot super in the usual way. If the assumed floor load represents the superimposed or "live" load, the weight of the floor must be added and the main girders proportioned to carry the whole of the load. The weight of Broad Flange Beam types of concrete floors may be taken as $\frac{3}{4}$ cwt. per square foot.* The table on page 142 enables suitable sizes to be selected without calculation for nearly 500 combinations of load, depth and span.

* Assuming that clinker-concrete is used, consisting of 1 part of cement to 2 of sand and 4 of clinker, the weight of the concrete when dry will be about 1 cwt. per cubic foot, and the total weight including steel and timber of the Broad Flange Beam types of floor (Figs. 1 to 3), if of a minimum depth of 5 inches, will be about 85 lbs., say $\frac{3}{4}$ cwt. per foot super. If the depth of the concrete is reduced to 4 inches, the weight will be reduced to about 75 lbs., say $\frac{2}{3}$ cwt. per foot super. If coke breeze concrete is used, consisting of 1 part of cement to 6 parts of fine breeze, the weight of the concrete when dry will hardly exceed 95 lbs. per cubic foot, and the weight of the floor will be reduced by about 10 lbs. per square foot. Ballast concrete behaves badly under fire, and is therefore unsuitable for floor construction—its weight is about 140 lbs. per cubic foot.

B.F. BEAM TYPES OF CONCRETE FLOORING.

Continued.

REINFORCING JOISTS.

These can be selected from the table on page 144 with equal ease.

The method of calculation is stated at the foot of the table. The formula adopted simply treats each joist as an ordinary beam with "fixed ends" carrying a distributed load, at $7\frac{1}{2}$ tons per square inch working stress, equal to its due proportion of the *superimposed* load.

This mode of calculation is very simple and entirely trustworthy. It is not theoretically the most correct. But the more scientific formulæ used in designing reinforced concrete pure and simple would be unsuitable in this case, because it is not intended to place confidence in the concrete to the extent which might be justified in the case of work designed and executed by experts in ferro-concrete.

In any case, over-refinement in calculation would be misplaced, as the pitch of the joists cannot be varied greatly and there are few suitable standard sizes of joists to choose from. It is recommended that these should be pitched as close as practicable; 28 inches is a convenient interval when timber joists are laid parallel to the steel joists. The most convenient sections are $3'' \times 1\frac{1}{2}''$ and $4'' \times 1\frac{3}{4}''$.* By increasing the pitch and using deeper joists than would otherwise be suitable, a little weight can be saved, but as the cost of the reinforcing joists averages only $1\frac{1}{2}$d. to 2d. per square foot of floor, this is no great advantage. On the other hand, the efficiency of the floor is seriously impaired if the pitch is increased much beyond 28 to 30 inches. If the thickness of the concrete above the joists is decreased, the floor is weakened and the joists may require to be spaced closer. The 2″ of concrete below the reinforcing joists is only dead weight and is intended solely for fire protection. If only partial resistance to fire is desired, the thickness may very well be reduced to one inch.

PROTECTION OF LOWER FLANGES OF MAIN GIRDERS.

It is recommended that expanded metal lathing or similar contrivance be bent round the lower flanges of the Broad Flange Beams as an additional safeguard to hinder the concrete from flaking or peeling off under fire (see Fig. 6 on page 148). Expanded metal $1\frac{1}{2}'' \times \frac{1}{8}'' \times 4$ lbs. per yard super is a convenient size.

The concrete under the main girders need not be more than $2\frac{1}{2}''$ thick. A good aggregate for this purpose is coke breeze: 5 parts coke breeze to 1 part of cement. The coke breeze to pass a $\frac{1}{2}''$ mesh.

The external angles of the encasements are preferably rounded off or splayed; but, even without this precaution, experiment shows that the encasements may be heated to incandescence and remain perfectly intact, if constructed as recommended above.

CONCRETE AGGREGATE.

For light floors, a coke breeze concrete is largely used, consisting of 1 part of Portland cement and 6 parts of fine breeze. For heavier floors, the following appear to be the most suitable aggregates :—

(1) 3 (or 4) parts of picked clinker and 2 of pit sand to 1 part of best Portland cement.

(2) 3 parts of common (stock) broken brick and two of pit sand to 1 part of best Portland cement.

The cement may be stipulated to pass the British Standard tests for Portland cement as published by the Engineering Standards Committee.

The "centering" may be removed after fourteen days and the load applied after twenty-eight days, in fine weather.

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* Or the corresponding metric standard sections.

FIRE TESTS ON BROAD FLANGE BEAM TYPES OF CONCRETE FLOORS.

The following account refers to two tests conducted by the British Fire Prevention Committee on floors of the type illustrated in Fig. 1, page 146.

It is reprinted, by permission, from the *Builders' Journal* of May 9th, 1906. It is to be observed that both floors were designed on the principles laid down in this chapter (pages 150 and 151) and were identical as regards size and arrangement of the girders and reinforcing joists, thickness of concrete etc. The test with Thames ballast concrete is interesting as showing the behaviour of the reinforcing joists, none of which were "dislodged, twisted or bent" (see official summary opposite), despite the severe way in which these were tested by the failure of the concrete in this case and consequent deflection of the supporting girders.

The Committee expressly prohibit the publication of unofficial reports by testors on the tests conducted by them or the reprint of complete copies of their official report. Consequently, readers who desire more detailed information concerning these two tests are requested to apply for copies of the official report.

CONCRETE FLOORS.

The object of the two experimental tests conducted by the British Fire Prevention Committee was to obtain data in respect to the fire-resistance of concrete floors supported by broad flange beams, the light reinforcing joists not being secured to the beams by cleats, bolts or other mechanical means, *i.e.* being held in position by the concrete between the wide flanges of the beams.

The idea was to see if such floors could obtain classification as being "fully protective" under the Universal Standards of 1903, *i.e.* whether they would withstand fire (from below) for four hours at temperatures ranging to about and over 1,800° Fahr., followed by water from a steam fire-engine for five minutes, the floors being loaded 2½ cwts. per foot super.

The sections of floors under test were the largest tested anywhere in the world on the fire issue under the standard requirements, measuring 334 feet super., the main beams being 15 feet span.

The conditions, except for some minor points, were identical except for the concrete aggregate, and the tests were classed as experimental tests in respect to joist and concrete floors, as they are not subject to patents, the materials being generally available.

The principles adopted in the design and construction of the floor are explained below.

As to the results, they bear out in every way the conclusions already arrived at by the Committee that Thames ballast concrete is entirely unreliable as a fire-resistant. The failures have been successive and without exception. The clinker-concrete stood remarkably well, and again bore out its excellent reputation. The tests on this large scale, following those on a smaller scale, will certainly naturally affect the building and fire regulations of the future as to the nature of concrete to be used in floors and for girder protection.

As to the floors under test, they measured $15' \times 22'\ 3''$, divided into three bays of 7 feet span centre to centre, the main girders being the "broad flange" type provided by Messrs. H. J. Skelton & Co.

The Thames ballast aggregate was 2 parts of washed sand, 4 parts of unscreened washed gravel and 1¼ parts Portland cement. The clinker-concrete was of 3 parts furnace clinker (¾ inch ring), 2 parts sand and 1 part Portland cement.

FIRE TESTS ON BROAD FLANGE BEAM TYPES OF CONCRETE FLOORS.—Continued.

[Reprinted from the "Builders' Journal."]

COMPARISON OF THE RESULTS.

The following results are in tabular form from the Committee's official summary, and show the failure of the Thames ballast concrete and the success of the clinker-concrete floors. We have added extracts from Mr. Marsland's preliminary notes to the two reports :—

The Thames Ballast Concrete Floor.

In 22 minutes after the commencement of the test the soffit of the concrete floor began to split off in patches, and continued to do so at intervals during the test.

In 75 minutes the whole of the concrete casing to lower flange of beam between south and centre bays fell.

In 100 minutes that to the beam between the north and centre bays fell.

The beams of the floor began to deflect in 20 minutes, and continued to do so more rapidly after the concrete to the lower flanges fell, till a maximum deflection of 7 3/10 inches (·18 m.) was recorded. Towards the end of the test the concrete to the north bay between the cross-joists began to fall and the wooden floor on top became ignited. At the conclusion of the test the lower flanges of the beams were seen to be red-hot.

On the application of water more of the soffit fell, and eventually nearly the whole of the concrete between the cross-joists in the north bay fell, the wooden floor on top igniting.

The maximum permanent deflection of the floor as recorded four days after the test was 4⅞ inches (·12 m.).

The concrete at the junction of the broad flange beams and the cross-joists did not crack or break. None of the joists were dislodged, twisted or bent.

Both fire and water passed through the floor, which was badly damaged.

Mr. Marsland's Note.

This test demonstrates the unreliability of ordinary gravel or Thames ballast concrete as a fire-resisting material at high temperatures.

It also demonstrates that with broad-flanged beams the connecting of the cross-joists to the main beams by cleats, bolts or rivets may be dispensed with when the concrete is carefully placed ; but such construction exposes a wider flange to the action of the fire, which requires careful protection.

The Clinker-Concrete Floor.

In 25 minutes after the commencement of the test small pieces of concrete began to split off soffit.

In 31 minutes two patches of concrete about 1 inch (·025 m.) thick became detached from soffit of south bay and fell.

The maximum deflection recorded was ·35 inch (·0088 m.) after 240 minutes.

On the application of water the surface of the beams was eroded where struck by the jet. (See illustrations.)

The soffits of the bays were also slightly eroded where struck by the jet.

On the load being removed the top surface of the wood floor was intact.

On removing the floor-boards some of them were discoloured by the heat on the underside, and a wood strip at the south-west corner was charred.

Fine diagonal cracks on the top of the floor extending about 3 feet were found at the south-west, south-east and north-west corners, also cracks on each side of the concrete over the tops of beams.

After the removal of the load it was found the floor was practically level and intact, and there was no permanent deflection.

Neither fire nor water had passed through the floor. Classification was obtained.

Mr. Marsland's Note.

It is interesting and instructive to compare this test with the former.

The floors were practically identical so far as their construction was concerned, the only difference being the aggregate of which the concrete to the bays and supporting beams was composed.

The test clearly demonstrates the superiority of clinker and coke-breeze over Thames ballast.

This test also again demonstrated that with broad flange beams the connecting of cross-joists to main beams by cleats, bolts or rivets may be dispensed with.

DESIGN OF THE FLOOR.

The floor, which is not subject to patent or special rights, was constructed of materials generally available. The following particulars of the floor, and details explaining the principles adopted in its design, have been prepared by Messrs. H. J. Skelton & Co. :—

(1) The steelwork consisted of rolled steel "broad flange beams" and light reinforcing joists. The joists were not secured to the beams by cleats, bolts or other mechanical means, but were secured by the concrete held between the wide flanges of the main beams, this being the characteristic feature of the floor under test.

(2) The floor was designed to carry a load, including the weight of the floor, of 392 lbs. per square foot ; this would give an average of about 20 tons on each of the broad flange beams. This load, according to the published properties of the section employed (viz. 10¼″ × 10¼″, of which the published Section Modulus about xx is

H 171 Foundations. Misc. Shapes. Misc. Tables. Prices etc. Code. Grey Mill. Tests etc. R.I.B.A. Photos

FIRE TESTS ON BROAD FLANGE BEAM TYPES OF CONCRETE FLOORS.—Continued.

[Reprinted from the "Builders' Journal."]

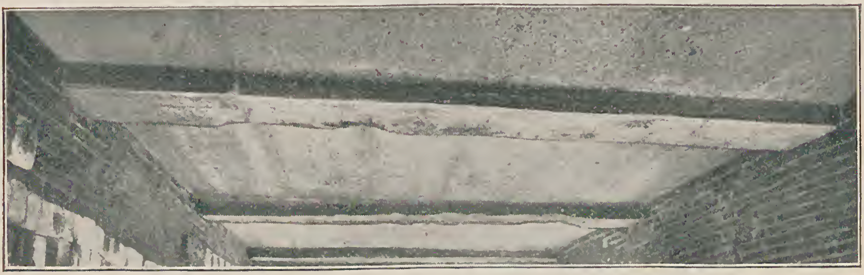
67 inches[3]), is equivalent to an extreme fibre stress of approximately $7\frac{1}{2}$ tons per square inch for a beam supported at both ends and uniformly loaded.

The thickness of the concrete was fixed at a minimum of 5 inches in accordance with building regulations. For the purpose of eliciting the required number and size of the reinforcing joists, the weight of the superimposed load alone was taken into account, the floor being assumed to carry its own weight. The joists were so spaced as to make the average load on each such as would impose an extreme fibre stress of $7\frac{1}{2}$ tons per square inch on the assumption of fixed ends. (Section Modulus about xx 1·835.)

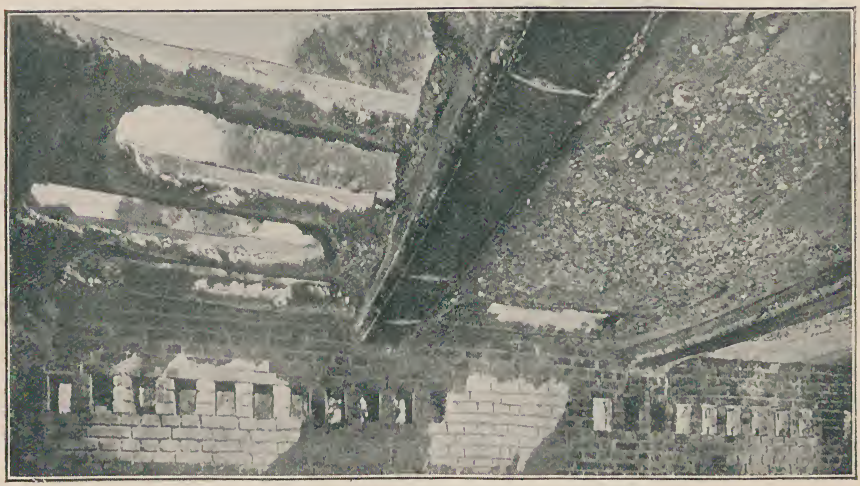
In making this calculation the span was measured in the clear, *i.e.* between the edges of the flanges of the main beams, on account of the assumption of fixed ends, whereas the span of the broad flange beams was measured from centre to centre of the wall-bearing at either end.

The weight of the steelwork in this floor, per bay, was 1.160 lbs., which works out at 11 lbs. per foot super. The extreme depth of floor was $14\frac{1}{2}$ inches.

(3) As mentioned in the report, the ends of each steel beam were connected longitudinally to the adjoining beam by tie-rods. It should be here pointed out, however, that the designers regard the provision of tie-rods in this type of floor as unnecessary and of no advantage as regards strength.



The Clinker-Concrete Floor that obtained "full classification."



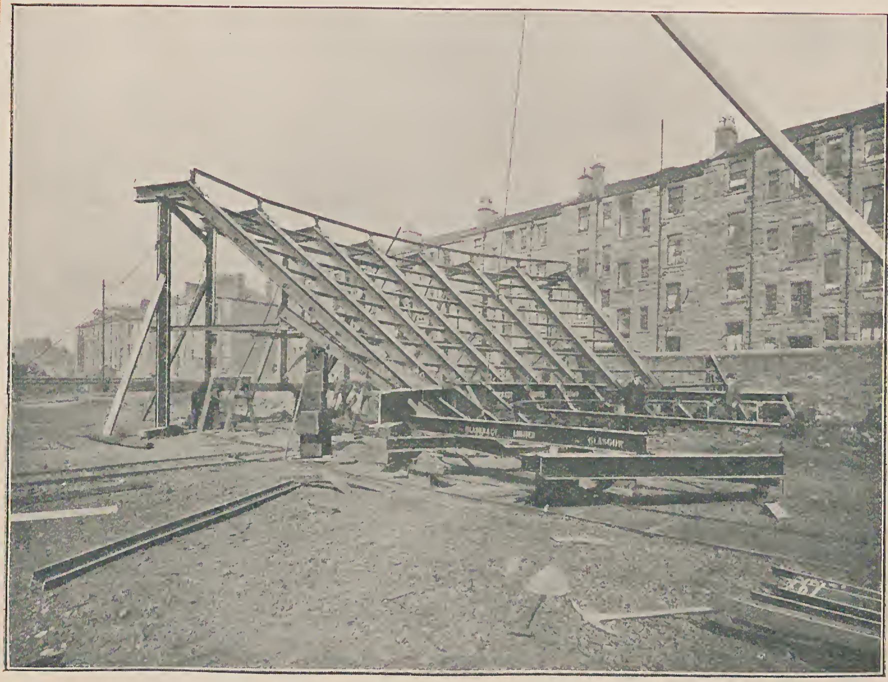
The Thames Ballast Concrete Floor that failed.

BROAD FLANGE BEAMS IN THEATRE CONSTRUCTION.

STEELWORK FOR "GRAND THEATRE AND OPERA HOUSE," FALKIRK, N.B.

Architect:
ALEXANDER CULLEN, Esq., F.R.I.B.A., Hamilton, N.B.

Structural Engineers:
Messrs. BLADEN & CO., Limited, Glasgow.



The above photograph shows the structural work of the Gallery. The Main Girder is of section $26'' \times 12''$, the Side Girders of section $20'' \times 12''$, the Raking Girders of section $13\frac{1}{2}'' \times 12''$, and the Side Auxiliaries of section $9\frac{1}{2}'' \times 9\frac{1}{2}''$. By using Broad Flange Beams for the Main Girder and Side Auxiliaries, the expense of riveted girders is avoided; while, in the case of the Raking Girders, valuable headroom is saved on account of the great carrying power of Broad Flange Beams combined with relatively shallow depth.

The Dress Circle of the above theatre was constructed entirely of Broad Flange Beams in similar fashion. The Roof over the Auditorium was also supported wholly on Broad Flange Beams, except for a deep main girder 5 feet in depth.

Messrs. Bladen & Co., who are specialists in this class of work, have constructed the steelwork of other theatres in similar fashion.

H
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Foundations.
Misc. Shapes.
Misc. Tables.
Prices etc.
Code.
Grey Mill.
Tests etc.
R.I.B.A.
Photos.

BROAD FLANGE BEAMS AS STANCHIONS.

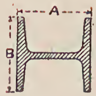


TABLE A. FIDLER'S FORMULA (FACTOR OF SAFETY 5 : 1).

This Table of Safe Loads is specially adapted to all ordinary building requirements (see Notes below).

Size. A × B	Weight per foot.	Sectional Area.	Least Radius of Gyration.	12 ÷ least Radius of Gyration.	Section No.	Coefficients for Eccentric Loads. Flange.	Coefficients for Eccentric Loads. Web.	Safe Loads in Tons. 8′	9′	10′	11′	12′	13′
Inches.	Lbs.	Sq. Ins.	Ins.										
7 × 7	31½	9·3	1·67	7·2	104	2·38	1·21	43	42	40	38	36	34
8 × 8	37	10·9	1·86	6·5	108	2·36	1·19	52	51	49	47	45	43
8½ × 8½	44	12·8	2·04	5·9	112	2·35	1·18	62	61	59	57	55	53
9½ × 9½	51	15·0	2·21	5·4	116	2·36	1·19	74	72	71	69	67	65
10 × 10	55	16·3	2·30	5·2	120	2·36	1·19	82	79	78	76	74	71
10¼ × 10¼	61	17·9	2·39	5·0	124	2·36	1·19	91	88	86	84	83	80
10½ × 10½	65	19·1	2·49	4·8	128	2·36	1·19	97	95	93	91	89	87
11 × 11	70	20·4	2·58	4·6	132	2·36	1·19	104	103	100	97	96	94
11½ × 11½	75	21·9	2·66	4·5	136	2·36	1·19	111	110	107	105	103	101
12 × 12	80	23·6	2·76	4·3	140	2·36	1·19	120	119	117	114	112	110
12½ × 12	85	24·9	2·75	4·4	144	2·37	1·20	128	125	123	120	118	116
13½ × 12	88	25·9	2·74	4·4	148	2·37	1·21	132	131	129	125	123	121
14 × 12	96	28·1	2·74	4·4	152	2·38	1·22	143	142	139	136	133	130
15 × 12	101	29·6	2·73	4·4	156	2·39	1·23	151	150	146	143	140	138
16 × 12	107	31·6	2·72	4·4	160	2·41	1·24	161	159	155	152	149	147
17 × 12	113	33·2	2·70	4·4	164	2·42	1·26	169	167	163	160	157	154
18 × 12	121	35·6	2·68	4·5	168	2·44	1·28	181	180	175	171	168	165
19 × 12	128	37·5	2·67	4·5	172	2·44	1·29	191	189	184	181	177	174
20 × 12	138	40·6	2·63	4·6	176	2·47	1·32	206	204	199	195	191	188
22 × 12	152	44·7	2·60	4·6	180	2·49	1·35	226	223	218	213	209	206
24 × 12	158	46·6	2·56	4·7	184	2·51	1·37	236	233	227	222	218	214
26 × 12	166	48·7	2·51	4·8	188	2·53	1·39	247	242	237	232	227	222
30 × 12	177	52·0	2·43	4·9	196	2·56	1·42	263	256	251	246	241	234

NOTES TO THE ABOVE TABLE.

This table has been drawn up to meet the requirements of those who desire to have tables specially adapted to the conditions obtaining in ordinary building construction. The margin of safety is ample without being extravagant and, unless exceptional conditions exist, such as special vibration or shock of machinery etc., this table can be confidently used in selecting stanchions for building work without special allowances or calculations. These safe loads allow a somewhat greater margin of safety than the allowances proposed by the Royal Institute of British Architects (see pages 232 and 233).

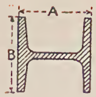
ECCENTRIC LOADS.

The coefficients given in the above table are used as follows :—Multiply the load (or that part of it which is eccentric) by the coefficient for eccentric loading. The result gives the centrally applied load to which the eccentric load is equivalent, If the eccentric load is applied by a girder connected to one of the flanges of the stanchion, use the coefficient in the column headed "Flange." If the connection is to the web of the stanchion, use the "Web" coefficients. This subject is more fully treated on pages 171 to 174, where the *modus operandi* is explained in greater detail.

CAST-IRON COLUMNS, BUILT-STANCHIONS Etc. See pages 163 etc.

BROAD FLANGE BEAMS AS STANCHIONS.

TABLE A.—Continued.



Size.		SAFE LOADS IN TONS.—Continued.												
A	B	14′	15′	16′	17′	18′	19′	20′	22′	24′	28′	32′	36′	40′
Inches. 7	× 7	31	29	27	25	24	22	20	18	15	...	...	...	...
8	× 8	41	38	36	34	32	30	28	24	21	...	...	...	...
8½	× 8½	51	49	46	43	41	39	36	32	29	23	...	...	...
9½	× 9½	63	61	58	55	52	49	47	42	38	30	25	...	...
10	×10	69	67	65	62	59	55	53	47	43	35	28	...	...
10¼	×10¼	77	75	73	70	68	65	61	55	50	41	33	28	...
10½	×10½	84	81	79	77	74	71	68	61	55	45	38	31	...
11	×11	91	88	86	84	81	78	75	68	62	51	42	35	...
11½	×11½	99	96	93	91	88	85	83	75	69	57	47	39	...
12	×12	108	105	102	99	97	94	91	85	77	65	54	45	39
12½	×12	114	111	108	105	102	99	96	89	81	68	57	48	41
13½	×12	119	116	112	109	106	103	100	92	84	70	58	49	42
14	×12	129	125	121	118	115	112	108	100	90	75	63	53	46
15	×12	135	131	128	124	121	118	114	104	96	80	67	55	48
16	×12	145	140	136	132	129	125	121	111	101	85	70	59	50
17	×12	151	147	143	139	135	131	126	116	106	88	73	61	52
18	×12	162	157	153	148	144	139	135	123	112	94	78	65	55
19	×12	170	165	161	156	152	147	142	129	118	98	81	68	58
20	×12	184	178	172	168	163	158	152	136	125	105	86	71	...
22	×12	200	194	188	184	178	172	166	149	136	113	93	78	...
24	×12	207	202	195	190	185	178	170	153	139	116	95	80	...
26	×12	215	209	202	197	190	183	176	157	143	117	97	81	...
30	×12	226	219	213	206	198	191	178	163	147	120	98	82	...

NOTES TO THE ABOVE TABLE.—Continued.

MODE OF CALCULATION.

The tabulated loads represent one-fifth of the calculated destructive loads, the latter having been estimated on the basis proposed by Mr. Fidler in his paper No. 2170, Proceedings of the Institution of Civil Engineers, Vol. 86, page 261. Mr. Fidler's formula is the Euler formula modified in accordance with an assumed deviation, such as occurs in every actual strut, from theoretical perfection in the matter of the centre of resistance of its section.

This deviation is practically equivalent to an initial set or deflection; and its amount and effect are estimated partly on mathematical theory and necessarily also in part on empirical data. The latter were supplied by the well-known experiments of Mr. James Christie at the Pencoyd Iron Works, Pennsylvania, in 1884. The destructive loads assumed in compiling this table correspond with those published by Mr. Fidler for columns with "fixed" ends, except that they have been modified to suit an assumed ultimate strength of 60,000 pounds (26·8 tons) per square inch.

COMPARATIVE TABLE OF STRESSES.

The table on page 166 shows how the above loads compare with those corresponding to various other formulæ.

To ascertain ratio of height (inches) to least radius of gyration, see page 175, or multiply the figures headed 12/r in the above table by the height of the stanchion in feet.

To ascertain compressive stress corresponding to any safe load in above table, see page 175.

BROAD FLANGE BEAMS AS STANCHIONS.

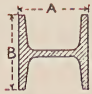


TABLE B. FIDLER'S FORMULA (SLIDING FACTOR OF SAFETY).

Maximum Safe Loads for Stanchions with both ends securely fixed. Load stationary and centrally applied (see Notes below). For Stanchions in ordinary building construction, use Table A.

Size.		SAFE LOADS IN TONS.															
A	B	10′	11′	12′	13′	14′	15′	16′	17′	18′	19′	20′	24′	28′	32′	36′	40′
Inches.																	
7 ×	7	52	49	46	43	40	37	34	31	29	26	24	17	...	...	...	...
8 ×	8	65	62	58	55	53	49	46	42	39	36	33	25	...	...	...	...
8½ ×	8½	80	77	73	69	66	63	59	55	52	48	45	34	26	...	...	...
9½ ×	9½	98	93	89	86	81	78	74	70	66	63	59	45	35	27	...	...
10 ×	10	108	103	99	95	90	87	82	79	75	71	67	52	41	32	...	...
10¼ ×	10¼	120	116	111	107	103	98	94	90	86	81	77	61	48	38	30	...
10½ ×	10½	129	126	120	116	112	107	103	99	94	90	85	68	55	44	34	...
11 ×	11	140	136	131	126	122	118	112	108	104	100	95	78	62	50	39	...
11½ ×	11½	151	147	143	137	133	128	124	118	114	109	105	87	69	56	45	...
12 ×	12	164	160	155	151	145	141	136	131	126	121	117	98	79	64	53	42
12½ ×	12	173	169	164	158	153	148	144	138	133	128	123	103	83	68	56	44
13½ ×	12	180	175	170	165	159	154	149	144	138	134	127	106	86	69	57	46
14 ×	12	195	190	185	179	173	167	162	156	150	145	138	115	93	75	62	50
15 ×	12	205	200	195	188	182	176	170	163	157	152	145	121	97	79	64	52
16 ×	12	219	213	207	200	193	187	180	173	167	160	154	128	103	83	68	55
17 ×	12	230	223	217	209	202	197	189	181	174	167	161	134	110	86	71	57
18 ×	12	245	238	232	225	216	209	202	194	186	178	172	142	114	92	75	60
19 ×	12	259	251	245	235	227	220	212	203	196	188	181	149	119	97	78	63
20 ×	12	279	272	264	253	245	236	227	218	209	201	193	156	126	103	82	...
22 ×	12	306	297	288	277	268	258	248	239	228	219	210	171	137	110	88	...
24 ×	12	318	308	299	287	277	268	256	246	236	227	214	173	140	112	88	...
26 ×	12	332	322	309	298	287	276	266	254	243	232	221	177	141	114	89	...
30 ×	12	349	338	325	313	300	287	276	264	253	239	227	182	143	114	89	...

NOTES TO THE ABOVE TABLE (B).

The above safe loads are considered to be the maxima that can be safely applied under the most favourable conditions, *e.g.* in a genuine steel-framed building constructed of wide-flanged material throughout, with first-class connections and workmanship, provided that the loads are practically stationary. It is recommended that Table A should be used in ordinary circumstances and the above table only used by fully qualified engineers.

The above loads are based on the same assumed destructive loads as in the previous table (A), where the mode of calculation is explained. In the above table, however, the factor of safety varies from $3\frac{1}{3}$ for $l/r=0$ to 4 for $l/r=100$ and 5 for $l/r=200$, the formula being :—

$$\text{Safe Load} = \text{Destructive Load} \times (\cdot 3 - \cdot 0005\ l/r).$$

This is a slight modification of the "Pencoyd" factor (see page 169), the object being to reduce the maximum unit stress for $l/r=0$ to one-half of the assumed limit of elasticity (16 tons per square inch).

COMPARATIVE TABLE OF STRESSES.

The table on page 166 shows the safe stresses for the various values of l/r and enables comparison with the stresses allowed by various other formulæ.

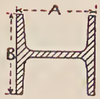
To ascertain ratio of height to least radius of gyration, or stress per square inch corresponding to above loads, see page 175.

ECCENTRIC LOADS.

Multiply the load, or that part of it which is eccentric, by the coefficients for eccentric loading as given in Table A, page 156.

BROAD FLANGE BEAMS AS STRUTS.

TABLE C. EULER'S FORMULA (PIN-ENDED STRUTS).
FACTOR OF SAFETY: 5. See Notes below.
For Stanchions in building construction, use Table A.



Size.		SAFE LOADS IN TONS.															
A	B	10′	11′	12′	13′	14′	15′	16′	17′	18′	19′	20′	24′	28′	32′	36′	40′
Inches.																	
7	7	46	38	32	27	23	20	18	16	14	13	11	8	...	...	...	...
8	8	65	55	46	39	34	30	26	23	20	18	17	11	...	...	...	...
8½	8½	77	77	65	56	48	42	37	32	29	26	24	16	12	...	...	...
9½	9½	90	90	90	76	66	57	50	44	40	35	32	22	16	12	...	...
10	10	98	98	98	90	77	67	59	52	47	42	38	26	19	15	...	...
10¼	10¼	108	108	108	106	92	80	70	62	56	50	45	31	23	17	14	...
10½	10½	115	115	115	115	105	92	81	72	64	57	52	36	26	20	16	...
11	11	123	123	123	123	121	106	94	83	74	66	60	42	30	23	18	...
11½	11½	131	131	131	131	131	120	105	94	84	75	68	47	35	26	21	...
12	12	141	141	141	141	141	140	123	110	98	87	79	55	40	31	24	20
12½	12	149	149	149	149	149	147	129	115	102	92	83	58	42	32	26	21
13½	12	156	156	156	156	156	151	133	119	105	94	86	59	44	33	26	21
14	12	169	169	169	169	169	164	144	129	114	102	93	64	47	36	29	23
15	12	178	178	178	178	178	171	151	134	119	107	97	67	49	38	30	24
16	12	189	189	189	189	189	182	160	142	126	113	102	71	52	40	32	26
17	12	199	199	199	199	199	188	166	149	131	117	106	74	54	42	33	27
18	12	213	213	213	213	213	199	175	156	138	124	112	78	57	44	35	28
19	12	225	225	225	225	225	207	183	163	144	130	117	82	60	46	36	30
20	12	243	243	243	243	243	218	193	171	152	137	123	86	63	48	38	...
22	12	268	268	268	268	268	235	206	184	163	147	132	92	68	52	41	...
24	12	280	280	280	280	273	238	209	185	165	148	134	93	68	52	41	...
26	12	293	293	293	293	275	239	210	187	166	149	135	94	69	53	42	...
30	12	312	312	312	312	275	239	210	187	166	149	135	94	69	53	42	...

NOTES TO THE ABOVE TABLE (C).

The above table is calculated in accordance with Euler's formula for elastic bending as applied to pin-ended struts.

Namely :—Crippling Load $= \Pi^2 E I \div l^2$.

$E =$ Modulus of Elasticity. $I =$ Least Moment of Inertia.

Taking E as 12,800 tons per square inch and allowing a factor of safety of 5, the safe pressure per square inch becomes:—$P = (25{,}260 \times r^2) \div l^2$, where $r =$ least radius of gyration (inches).

As Euler's formula only applies to *long* columns which bend under the load before the limit of elasticity of the material is reached, it is necessary to assign, for short columns, a maximum stress per unit of sectional area, based on the ultimate strength or limit of elasticity of the material employed. In the above table, the maximum stress allowed is 6 tons per square inch, this applying to sizes or lengths for which the l/r is less than 65.

MODIFICATIONS FOR END-FIXING.

For struts with one end fixed, the length of the strut (l) is taken as two-thirds of the actual height or length ; if both ends are fixed, l is taken as one-half the actual length ; if one end is fixed and the other entirely free (not pivoted), the length is taken as four times the actual height or length.

ECCENTRIC LOADS.

Multiply the load, or that part of it which is eccentric, by the coefficients given in Table A, page 156.

For Comparative Table and Diagram of Stresses, see page 166.
For Values of l/r and stresses corresponding to above loads, see page 175.

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Foundations.
Misc. Shapes.
Misc. Tables.
Prices etc.
Code.
Grey Mill.
Tests etc.
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Photos.

USEFUL TYPES OF BUILT-STANCHIONS COMPOSED OF BROAD FLANGE BEAMS.

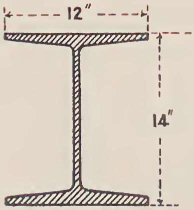

Fig. 1

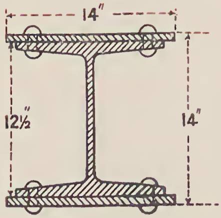

Fig. 2

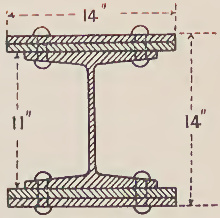

Fig. 3

Figs. 1 to 3 show the facility with which Broad Flange Beams can be plated for use as stanchions or girders of varying strength but of uniform depth, when this is desired. (See also Table of Safe Loads on Plated Beams and accompanying notes, page 185.)

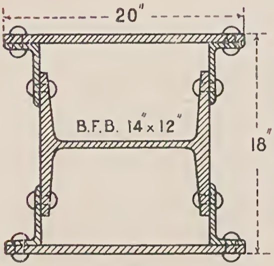

Fig 4

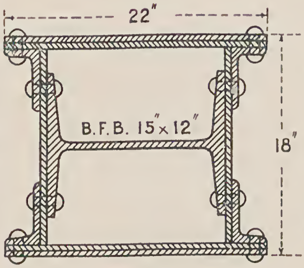

Fig. 5

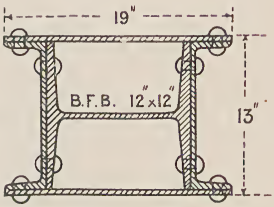

Fig. 6

Figs. 4 to 6 show stanchions composed of Broad Flange Beams with angles or channels riveted to the flanges. Type 6 can be constructed with all sizes of Broad Flange Beams.

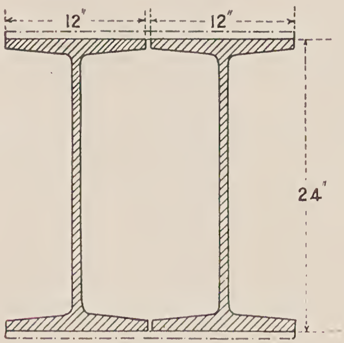

Fig. 7

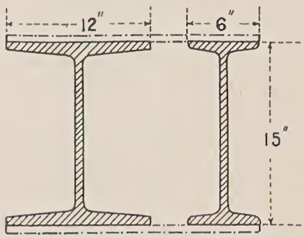

Fig. 8

Figs. 7 and 8 show double stanchions with plates riveted to the flanges or latticed with plates or angles. In Fig. 8, the wide-flanged stanchion carries a crane-bearing girder, while the joist section of equal depth is extended to carry the roof. This type can be constructed with all sizes of Broad Flange Beams up to 22″ × 12″.

USEFUL TYPES OF BUILT-STANCHIONS COMPOSED OF BROAD FLANGE BEAMS.—Continued.

For carrying very heavy loads, stanchions made up of two deep sections of Broad Flange Beams are remarkably efficient. For example, two beams 24″ × 12″ placed side by side (as in Fig. 5, page 181) make an excellent stanchion 2 feet square, which compares most favourably in economy and convenience with all other designs of equal strength. Stanchions of this type without flange plates can be devised to carry the heaviest loads ever found in ordinary building construction, *e.g.* the loads on the basement stanchions in an 18-story warehouse. Detailed comment on these built-up sections is unnecessary, because it will be evident that the advantages of wide-flanged sections in scientific distribution of weight, in flange area available for bolts and rivets etc., are such that when particularly heavy stanchions are required, sections can be built-up from Broad Flange Beams which will possess many advantages over other built-sections similar to the advantages which a plain Broad Flange Beam offers as compared with narrow-flanged joists and built-sections. [See, for example, sketch of a post in a triangulated girder, page 238.]

Broad Flange Beams (22″ × 12″) as Stanchions in Basement of Warehouse.

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Foundations.
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Prices etc.
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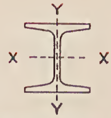
Photos.

SAFE LOADS ON BROAD FLANGE BEAMS AS STRUTS,

When liable to failure in the direction YY only.



Size.	Maximum Radius of Gyration (XX).	Sectional Area.	Safe Loads in Tons for heights of 10 to 44 feet.												C
			10	12	14	16	18	20	24	28	32	36	40	44	
Inches.	Ins.	Sq. Ins.													
7 × 7	3·01	9·3	47	45	43	42	40	38	33	28	24	20	17	15	2·38
8 × 8	3·38	10·9	56	54	52	51	49	47	43	37	32	28	24	21	2·36
8½ × 8½	3·72	12·8	66	65	63	61	59	57	53	48	42	37	32	28	2·35
9½ × 9½	4·05	15·0	78	77	75	73	71	70	65	60	55	48	42	38	2·36
10 × 10	4·22	16·3	84	83	82	80	78	76	71	66	61	54	48	43	2·36
10¼ × 10¼	4·38	17·9	93	92	91	89	86	84	80	75	70	62	56	50	2·36
10½ × 10½	4·56	19·1	99	98	97	95	93	91	87	81	76	69	62	55	2·36
11 × 11	4·73	20·4	106	105	104	103	100	98	94	88	83	77	68	61	2·36
11½ × 11½	4·90	21·9	114	113	112	110	107	105	102	96	90	84	76	69	2·36
12 × 12	5·07	23·6	123	122	121	120	117	115	110	105	98	93	85	77	2·36
12½ × 12	5·39	24·9	131	130	128	127	126	122	118	114	107	102	95	86	2·37
13½ × 12	5·71	25·9	137	136	134	133	132	129	125	121	114	109	103	96	2·37
14 × 12	6·02	28·1	148	147	146	145	143	142	136	132	127	120	115	108	2·38
15 × 12	6·33	29·6	156	155	154	153	152	151	145	141	136	130	123	118	2·39
16 × 12	6·63	31·6	167	166	165	164	162	161	156	151	147	141	135	129	2·41
17 × 12	7·03	33·2	176	175	173	172	171	170	166	160	157	152	146	139	2·42
18 × 12	7·39	35·6	189	188	186	185	184	183	180	174	169	165	159	153	2·44
19 × 12	7·79	37·5	200	198	197	196	195	193	191	185	180	176	171	165	2·44
20 × 12	8·12	40·6	216	215	213	212	211	210	207	204	198	192	188	181	2·47
22 × 12	8·86	44·7	238	237	235	234	233	232	230	227	222	215	211	206	2·49
24 × 12	9·62	46·6	249	248	247	246	245	243	241	238	235	229	224	219	2·51
26 × 12	10·4	48·7	260	259	259	258	256	255	253	250	248	243	238	233	2·53
30 × 12	11·8	52·0	278	278	277	276	274	274	272	270	267	265	261	256	2·56

C = Coefficient for an eccentric load applied to one flange.

NOTES TO THE ABOVE TABLE OF SAFE LOADS.

Stanchions are sometimes built into a brick wall or otherwise braced laterally in such a way as to be secured against failure in the direction of the flanges and to be solely liable to flexure in the direction YY (see sketch at head of table).

The above table is suitable in such cases, the safe loads having been calculated with respect to the greatest radii of gyration, *i.e.* the radii of gyration about the axis XX. In other respects the basis of calculation is the same as that of the ordinary table of safe loads (viz. Table A, page 157).

ECCENTRIC OR UNBALANCED LOADS.

The coefficients in the column headed "C" are used as follows:—Multiply the load (or that part of it which is eccentric) by the coefficient for eccentric loading. The result gives the centrally applied load to which the eccentric load is equivalent.

COMPARATIVE VALUES OF DIFFERENT TYPES OF STANCHIONS.

CAST-IRON COLUMNS.

The use of cast-iron in bridge-work has been practically abandoned, and it is being rapidly given up in building work also.

The advantages of cast-iron are its great strength in compression and its cheapness, which often enables a comparatively ornamental column of cast-iron to be employed at the same price as a steel column, despite the extra weight.

On the other hand, there are serious objections to the use of cast-iron columns.

(1) When castings have to be bored etc. for use in machinery, blow-holes are not uncommonly encountered and the casting is rejected. But in the case of columns for building construction, flaws of this kind are less likely to be detected.

(2) The molten iron has frequently to be poured into the mould from both ends; so this, in the case of a cast-iron column, often results in weakness at the centre where strength is most required.

(3) Hollow cast-iron columns are sometimes thicker on one side than on the other, owing to the core shifting in the process of casting.

(4) It is practically impossible to ensure uniform cooling of the finished casting. Consequently, there is irregular contraction and internal stresses are caused which may render a column quite unsafe, no matter how high the factor of safety adopted in calculations.

(5) If more than one tier of cast-iron columns is employed, the columns have to be provided with lugs or flanges both to bolt the upper column to and also for bolting to the intermediate floor girders. These lugs or flanges are features of great weakness on account of their feeble resistance to bending strains, frequent "honeycombing" in them, and also because it is practically impossible to make rigid connections to the floor beams. In the case of a column loaded on one side only, or unequally loaded on the opposite sides, there is a bending tendency which cast-iron is particularly ill-fitted to resist.

SOLID STEEL COLUMNS.

Columns made of round steel bars are sometimes used with advantage in building construction where economy of space is of paramount importance, as in the galleries in theatre construction.

In many such cases, however, a simple system of cantilevers might be devised, dispensing with columns altogether at little or no extra cost.

Theoretically and practically, solid steel columns are not economical; their weight and cost being considerably greater than the weight and cost of steel H sections or built-stanchions of equal strength. They also present a difficulty in providing connections to an upper tier of columns or to intermediate floor girders similar to that which attaches to cast-iron columns, and are therefore quite unsuitable for buildings of several stories.

BUILT-STANCHIONS.

There are numbers of available types of built-stanchions each of which has its own special uses, advantages and disadvantages. It would be outside the scope of this book to discuss these various types in detail, but brief comparison may be made with stanchions composed of Broad Flange Beams.

(1) The latticed channel type consists of two channels spaced at a suitable distance apart and tied together by diagonal lattice bars, usually flat bars. When the channels are placed with the flanges inside and are so spaced as to make the radius of gyration in the direction of the flanges equal to that about the axis perpendicular thereto, this type of stanchion is about the most economical in metal of all types with which Broad Flange Beams are likely to be compared. It is, however, impossible to effect any saving in cost by substituting a stanchion of this type for a Broad Flange Beam of equal strength, because the increased cost per ton of the latticed section outweighs the economy

in weight, and makes the latter stanchion a more expensive one; considering the great amount of workmanship involved in proportion to the scanty weight of materials, this fact is not surprising.

(2) Another type of stanchion, better suited for building construction, is composed of two channels and two plates riveted to the flanges, forming a box section. If the channels are spaced at the most effective distance apart, it is possible, in most cases, to design stanchions of this type equal in strength to those of the Broad Flange Beam type at about the same weight. In most cases, however, the Broad Flange Beam type would be rather lighter* and in all cases cheaper on account of the considerable difference in cost of manufacture.

(3) If a stanchion is made up from two joists with flange plates to carry the same load as a plated channel stanchion of the foregoing type, its weight and cost will invariably be greater. *A fortiori*, therefore, this type is more expensive and not so effective as the Broad Flange Beam type.

It should be needless to say that this comparison does not extend to built-stanchions designed to carry heavier loads than those for which Broad Flange Beam stanchions are suitable. Heavy sections of these box types, weighing over, say, 140 lbs. per foot, show a rapidly increasing efficiency, *i.e.* a rapidly increasing ratio of strength to weight, as the weight is increased. But, as these heavy sections cannot be replaced by any single Broad Flange Beam, there is no question of comparison between them.

(4) Stanchions composed of a single rolled steel joist with flange plates are convenient, but it is impossible to design any stanchion of this type to do the work of a single Broad Flange Beam without materially increasing the weight and still further increasing the cost.

(5) There are various other types of built-stanchions composed of plates, joists, channels, angles and Z bars, but these are all heavy types designed to carry much greater loads than Broad Flange Beams, and need not therefore be considered in this connection.

In the foregoing remarks, the only questions dealt with are those of economy in weight and cost. To make a complete comparison between Broad Flange Beams and built-stanchions, it is necessary to take other factors also into account, some of which may materially affect the cost of steelwork:—

(1) Accessibility for painting. Many types of built-stanchions cannot be painted on the inner surfaces after assemblage. Moreover, built-sections are always more liable to rust than plain rolled steel sections.

(2) Plain wide-flanged stanchions are particularly convenient in factories of all kinds, as brackets for shafting etc. have often to be attached to the stanchions, and this is obviously not an easy matter in the case of built-sections.

(3) The great strength of connections to wide-flanged girders and stanchions is an important factor as regards the general stability of any structure in which they are employed. Moreover, on this account, Broad Flange Beams, when used as stanchions in conjunction with wide-flanged girders well fastened, have an ultimate strength in excess of that of built-stanchions of nominally equal carrying power, because there is a nearer approach to the theoretical conditions of "fixed" ends.

* This and similar statements are based on careful examination of typical examples. The method of comparison is to select a given Broad Flange Beam section and calculate its strength as a stanchion of a given height, in accordance with any approved formula. Then a stanchion of whatever built-type it is desired to compare with it must be designed from available materials in such fashion as to show the same or approximately the same strength as the Broad Flange Beam section when tested by the same formula. The following rule is useful as a rough guide for comparing the strength of steel stanchions:—However different in shape or size, their strengths are approximately proportionate to the least moments of inertia of their respective sections. *E.g.* a section having a least moment of inertia of 100 is twice as strong, approximately, as a section having a least moment of inertia of 50. The longer the stanchions, the more nearly correct this rule becomes, and *vice versa*, as the moment of inertia is only a measure of the resistance to buckling of a relatively long column.

(4) Time and expense are saved in construction (*i.e.* in the shops) because the amount of labour is reduced to a minimum.

(5) Time is saved in erection on account of the simplicity of the connections and the greater ease with which plain rolled steel H sections can be handled and assembled. [See also notes on "Connections," pages 10 to 12.]

(6) Time is obviously saved in design also.

(7) Steel bases for wide-flanged stanchions need not be carried so far up the shaft as bases to built-stanchions, which is a convenience and economy in building construction (see page 16).

ROLLED STEEL JOISTS AS STANCHIONS.

It need hardly be said that Broad Flange Beams are incomparably superior to ordinary narrow-flanged joists for use as stanchions, since the superiority of a square-shaped section is manifest. Weight for weight, the carrying power of Broad Flange Beams (as stanchions) is usually about twice as great as that of ordinary joists of the same depth. Even the best ordinary sections, namely 9″ × 7″ and 10″ × 8″ can be replaced by Broad Flange Beams of greater strength and less weight. The following examples are instructive :—

A. 9″ × 7″ Joist.		9½″ × 9½″ Beam.	
Weight 58 lbs. per foot.		Weight 51 lbs. per foot.	
Least radius of gyration ..	1·65″	Least radius of gyration ..	2·21″
Safe Load on 10 feet	72 tons.	Safe Load on 10 feet	71 tons.

The two sections are of practically equal strength, but the 9″ × 7″ joist is nearly 14 % heavier—a very important difference in cost.

B. 9″ × 7″ Joist.		8½″ × 8½″ Beam.	
Weight 58 lbs. per foot.		Weight 44 lbs. per foot.	
Least radius of gyration ..	1·65″	Least radius of gyration ..	2·04″
Safe Load on 20 feet	36 tons.	Safe Load on 20 feet	36 tons.

As this time a comparatively long column has been chosen, the superior disposition of the metal in Broad Flange Beams becomes increasingly evident. Both sections carry the same load although the 8½″ beam is 24 % lighter. Taking the cost of either type of stanchion at, say, £10 per ton, this saving in weight is equivalent to a reduction in price of £2 8s. per ton. If 9½″ × 9½″ section were used, having a safe load on 20 feet of 47 tons as compared with the 36 tons safe load for the 9″ × 7″ section, the beam is seen to be over 30 % stronger than the joist although the latter is 14 % heavier. These figures are taken from the tables of safe loads on pages 156 and 188 respectively, both of these tables having been calculated on the same basis. Any other recognised formula would produce results similar in the main. For example, the Rankine formula given on page 166 (for square bearing) gives the following safe loads for stanchions 15 and 20 feet high :—

	15 feet.	20 feet.
10″ × 8″ Joist (70 lbs. per foot)	91·5 tons.	78·8 tons.
10½″ × 10½″ Beam (65 lbs. per foot)	93·1 tons.	81·7 tons.

This particular formula has been selected from the 22 formulæ on page 166 because of all others it is the *least* favourable to Broad Flange Beams as compared with ordinary joists.* Yet, even on this unfavourable basis of comparison, a stanchion of 10½″ × 10½″ section is shown to be materially stronger than a similar stanchion of 10″ × 8″ section, despite the fact that the latter is 5 lbs. per foot, *i.e.* nearly 8 %, heavier. As the 10″ × 8″ joist section has a considerably higher carrying power (as a stanchion) in proportion to its weight than any other standard joist section, it would be superfluous to give any further examples in proof of the superiority of Broad Flange Beams in this respect.

* On inspection of the figures on page 166 it is evident that this example of the Rankine formula makes least allowance for *shape* (as represented by the radius of gyration) and most allowance for *weight* (as represented by sectional area).

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STRENGTH OF STEEL STANCHIONS.

COMPARATIVE TABLE OF SAFE PRESSURES ACCORDING TO VARIOUS FORMULÆ.

[See also Diagram on opposite page.]

The headings denote values of l/r from 0 to 200, where l=height of column (inches) and r=radius of gyration (inches).
The figures in the table are the corresponding safe pressures (P) in tons per square inch.

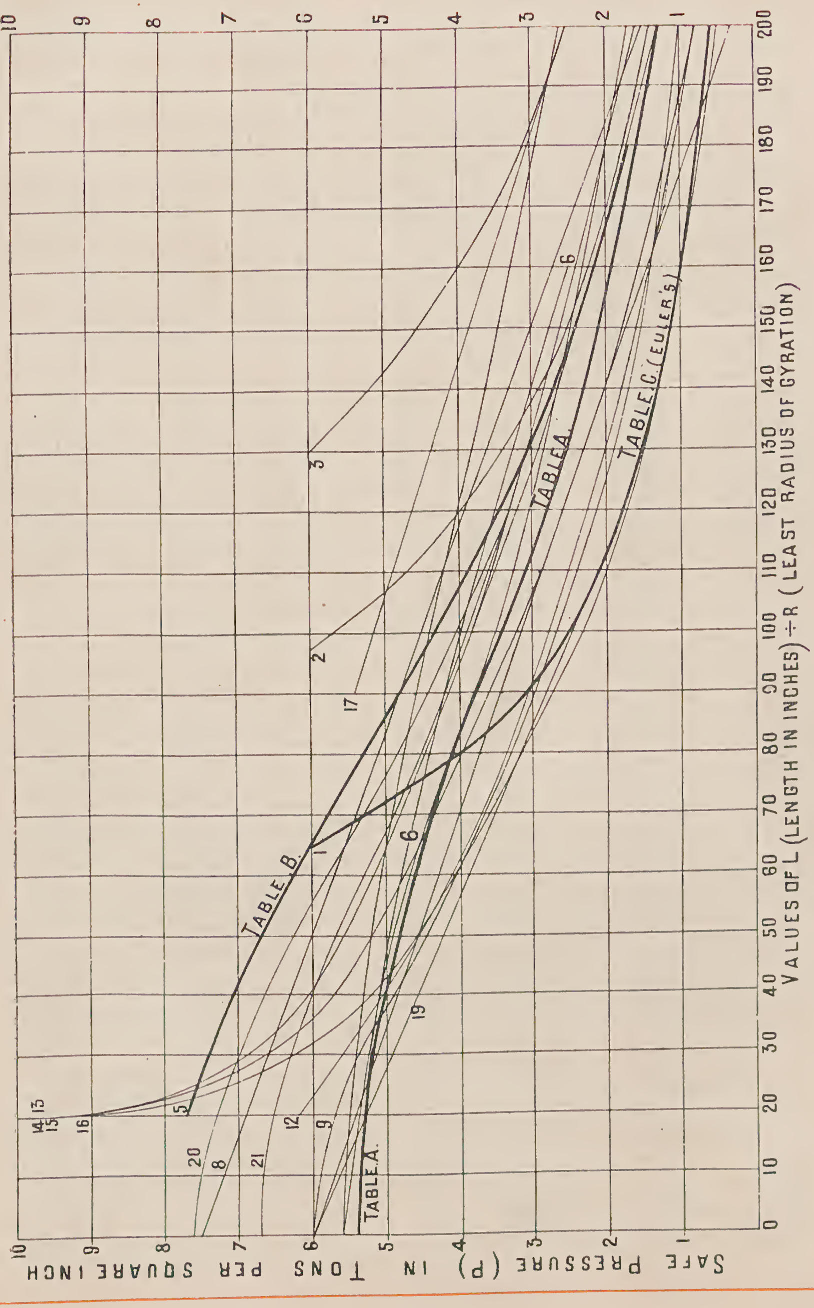
	FORMULA.	ENDS.	0	10	20	30	40	50	60	70	80	90	100	110	120	130	140	150	160	170	180	190	200
1	Euler's, Table C, page 159	pin	..	..	..	..	..	..	6·0	5·2	4·0	3·1	2·5	2·1	1·8	1·5	1·3	1·1	1·0	0·9	0·8	0·7	0·6
2	,, modified for one end flat or	fixed	..	..	..	..	..	..	..	..	..	6·0	5·7	4·7	4·0	3·4	2·9	2·5	2·2	2·0	1·8	1·6	1·4
3	,, ,, ,, both ends flat or	fixed	..	..	..	..	..	..	..	..	..	..	..	..	..	6·0	5·2	4·5	4·0	3·5	3·1	2·8	2·5
4	Table A on page 156	(fixed)	..	..	5·3	5·2	5·1	4·8	4·6	4·3	4·1	3·8	3·4	3·1	2·8	2·5	2·3	2·0	1·9	1·7	1·6	1·4	1·3
5	Table B on page 158	fixed	..	..	7·7	7·4	7·1	6·7	6·2	5·8	5·3	4·8	4·3	3·9	3·4	3·0	2·7	2·4	2·1	1·9	1·7	..	..
6	R.I.B.A. proposals	(fixed)	..	..	5·3	(5·2)	5·1	(5·0)	4·8	(4·6)	4·5	(4·2)	4·0	(3·7)	3·5	(3·2)	3·0	(2·7)	2·5	..	..	..	..
7	Straight Line. $P=6\,(1-\cdot00475\ l/r)$	(pin)	6·0	5·7	5·4	5·1	4·9	4·6	4·3	4·0	3·7	3·4	3·2	2·9	2·6	2·3	2·0	1·7	1·4	1·2	0·9	0·6	0·3
8	,, ,, $P=7\tfrac{1}{2}\,(1-\cdot004\ l/r)$	fixed	7·5	7·2	6·9	6·6	6·3	6·0	5·7	5·4	5·1	4·8	4·5	4·2	3·9	3·6	3·3	3·0	2·7	2·4	2·1	1·8	1·5
9	Rankine. $P=6\div\left\{1+\frac{1}{8000}\cdot\frac{l^2}{r^2}\right\}$	pin	6·0	5·9	5·7	5·4	5·0	4·6	4·1	3·7	3·3	3·0	2·7	2·4	2·1	1·9	1·7	1·6	1·4	1·3	1·2	1·1	1·0
10	Rankine (Carnegie Constants)	flat	(5·6)	(5·6)	(5·5)	5·4	5·3	5·2	5·1	4·9	4·7	4·6	4·4	4·2	4·0	3·8	3·6	3·4	3·3	3·1	2·9	2·8	2·6
11	,, ,, ,,	pin	(5·6)	(5·5)	(5·5)	5·3	5·1	4·9	4·7	4·4	4·1	3·8	3·6	3·3	3·1	2·9	2·7	2·5	2·3	2·1	2·0	1·9	1·7
12	Pencoyd - Wrought Iron	pin	..	..	6·2	5·6	5·0	4·5	4·1	3·7	3·4	3·1	2·8	2·5	2·3	2·0	1·8	1·6	1·4	1·2	1·0	0·9	0·8
13	,, - Mild Steel	fixed	..	..	9·8	6·9	6·0	5·6	5·2	4·8	4·5	4·1	3·8	3·6	3·4	3·1	2·9	2·7	2·5	2·2	2·0	1·8	1·6
14	,, ,, ,,	flat	.	..	9·8	6·9	6·0	5·6	5·2	4·8	4·5	4·1	3·8	3·5	3·2	2·9	2·6	2·4	2·1	1·9	1·7	1·5	1·3
15	,, ,, ,,	pin	..	..	9·5	6·6	5·7	5·2	4·8	4·4	4·0	3·6	3·2	2·9	2·6	2·3	2·0	1·8	1·5	1·3	1·1	0·9	0·8
16	,, ,, ,,	round	..	..	9·1	6·2	5·2	4·6	4·1	3·6	3·2	2·7	2·3	2·0	1·8	1·5	1·2	1·0	0·9	0·7	0·6	0·5	0·5
17	Carnegie - Built Columns	flat	..	..	..	..	..	..	..	..	..	5·4	5·1	4·8	4·6	4·3	4·1	3·8	3·6	3·3	3·1	2·8	2·5
18	Swiss Government	..	5·1	4·9	4·7	4·5	4·3	4·1	4·0	3·8	3·6	3·4	3·2	2·9	2·4	2·1	1·8	1·6	1·4	1·2	1·1	1·0	0·9
19	,, (Constants modified)	pin	6·0	5·6	5·3	4·9	4·6	4·2	3·9	3·5	3·2	2·8	2·5	2·1	1·8	1·5	1·3	1·1	1·0	0·9	0·8	0·7	0·6
20	Pencoyd (Railway Bridges) Mild Steel	..	(7·6)	7·4	7·3	7·0	6·6	6·2	5·7	5·3	4·8	4·4	4·0	3·6	3·3	..	..	..	..	..	..	..	..
21	,, ,, ,, Soft Steel	..	(6·7)	6·7	6·5	6·3	6·0	5·7	5·3	4·9	4·5	4·2	3·8	3·5	3·2	..	..	..	..	..	..	..	..
22	Dorman, Long & Co. (1906)	..	(6·0)	6·0	5·9	5·8	5·7	5·5	5·3	5·1	4·9	4·6	4·3	4·0	3·6	3·4	3·1	2·8	2·6	2·4	2·2	2·0	1·9

For explanation of the above table, see page 168.

STRENGTH OF STEEL STANCHIONS.

Diagram of Safe Pressures corresponding to Table opposite.

[For further explanation, see page 168.]



STRENGTH OF STEEL STANCHIONS.

Explanation of the Comparative Table and Diagram of Safe Pressures on pages 166 and 167.

To find the safe load which a steel column or strut of given shape and length (or height) will carry, in accordance with any one of the 22 tabulated formulæ*:—

Let l=height or length of strut (inches).

Let r=least radius of gyration of section (inches).

(N.B.—To calculate r, divide the Least Moment of Inertia by the Sectional Area and find the square root of the result.)

Let a=sectional area (square inches).

Let P=safe pressure (*i.e.* safe stress in compression) reckoned in English tons (2,240 lbs.) per square inch of sectional area.

The safe pressure (P) is determined by the ratio $l \div r$.

The table on page 166 shows the values of P, for various values of l/r from 0 to 200, according to each of 22 different formulæ.

Obviously: the Safe Load=Tabulated Safe Pressure × Sectional Area.

N.B.—The following reference numbers (1 to 22) are given in the table and diagram on pages 166 and 167.

(1) The first series is the basis of Table C, page 159, and is calculated by Euler's formula as applied to pin-ended struts. Taking a factor of safety of 5 and the modulus of elasticity of steel as 12,800 tons per square inch, the formula becomes:—$P=(25{,}260 \times r^2) \div l^2$. The maximum unit stress for short columns is taken as 6 tons per square inch. [For further notes, see page 159.] Euler's formula is the basis of all the various formulæ which have been proposed by different authorities for calculating the safe loads on steel struts. It is the formula most generally used on the Continent and also by many engineers in this country on account of its simplicity and mathematical correctness. But, for general practical use, some of the empirical formulæ are perhaps preferable.

(2) Euler's formula (No. 1) modified for struts with *one* end flat or fixed, the other pivoted, by taking l as two-thirds of the actual height or length.

(3) Euler's formula (No. 1) modified for struts with both ends fixed or flat, by taking l as one-half of the actual height or length.

(4) This series of safe stresses is the basis of Table A of Safe Loads on page 156, and represents one-fifth of the estimated destructive pressures, calculated by Fidler's formula (further explained on page 157). This formula was preferred because it appeared to be the most successful attempt yet made to bring Euler's theory into harmony with trustworthy experimental data.†

(5) This series of safe stresses is the basis of Table B of Safe Loads on page 158. The destructive pressures were calculated in the same manner as for Table A (Formula No. 4 above), but the factor of safety increases uniformly from $3\frac{1}{3}$ for $l/r=0$ to 5 for $l/r=200$. This series is intended to represent the maximum loads which may be safely applied to solid, wide-flanged steel struts under the most favourable conditions as to loading, fixing etc., and taking into account the practical superiority of such struts as compared with built-up or narrow-flanged sections of equal theoretical strength.

(6) Recommendations of the Royal Institute of British Architects for the Amendment of the London Building Acts. For copy of these, see page 232. The intermediate safe pressures for $l/r=30$, 50 etc., which were not published by the Institute, have been calculated by simple proportion and inserted for the sake of

* The comparison here instituted is not one of merely academic interest. Published tables of safe loads cannot be used with confidence unless accompanied by such information as will enable the reader to satisfy himself of their correctness. There is a single recognised formula, in use all over the world, for calculating safe loads on girders, and it is sufficient to state this formula and the working stress adopted. But there are many recognised formulæ for calculating safe loads on stanchions or struts. Consequently, it is well-nigh impossible to establish the correctness of a given table of safe loads on struts otherwise than by showing that the pressures allowed for given values of l/r accord with those allowed by the best authorities.

† See also Mr. Moncrieff's interesting essay on the "Practical Strength of Columns or Struts of Wrought-Iron and Mild Steel" in *Engineering*, 6th June, 1902.

uniformity. (In other instances where figures in the table are enclosed in brackets, a similar explanation applies.)

(7) "Straight line" formula.

$$P = 6\,(1 - 0{\cdot}00475\ l/r).$$

This type of formula is at present much used in England and America on account of its simplicity.

(8) This is another example of the straight line formula.

The figures tabulated represent one-fourth of the destructive pressures calculated by the formula $30 - 0{\cdot}12\ l/r$. This appears to be the basis of the table of "Calculated Breaking Loads in Tons, of Joists, used as Columns with ends fixed," published by Messrs. Dorman, Long & Co. Limited (Handbook, 1900).

The *safe* stresses given above are for quiescent loads: "The working load should not exceed one-fourth for stationary loads and one-sixth for moving loads."* [It will be observed that P vanishes for $l/r = 250$.]

(9) This is a form of the Rankine-Gordon formula as suggested in a previous edition.

$$P = 6 \div \left\{ 1 + \frac{1}{8{,}000} \frac{l^2}{r^2} \right\}$$

The origin of the Rankine formula is the inconvenience of having to distinguish between long and short columns and the desire to have one formula for all lengths.

The Rankine type of formula is used largely in this country and in the United States.

(10) (11) These are calculated from the following variations of the Rankine formula as adopted by the Cambria Steel Co. of Pennsylvania and the Carnegie Steel Co. The formulæ apply to "medium" steel.

P' denotes the destructive pressure in pounds per square inch.

Square bearing: $$P' = 50{,}000 \div \left\{ 1 + \frac{1}{36{,}000} \frac{l^2}{r^2} \right\}$$

Pin and square bearing: $$P' = 50{,}000 \div \left\{ 1 + \frac{1}{24{,}000} \frac{l^2}{r^2} \right\}$$

Pin bearing: $$P' = 50{,}000 \div \left\{ 1 + \frac{1}{18{,}000} \frac{l^2}{r^2} \right\}$$

"To obtain safe unit stress:—For quiescent loads as in buildings, divide by 4. For moving loads as in bridges, divide by 5."

The safe pressures (P) tabulated above as (10) and (11) represent one-fourth of the destructive loads calculated by the first and third of these formulæ respectively.

(12) (13) (14) (15) (16) The five series of safe pressures Nos. 12 to 16 are those adopted by the American Bridge Co. (Pencoyd Iron Works) on the basis of the well-known Christie experiments. The figures were compiled from the American Bridge Co.'s Handbook (1900). Series No. 12 represents the safe pressures for "Struts of Wrought-Iron or Extreme Soft Steel" with "Hinged" (*i.e.* Pivoted) Ends. These are given for the sake of comparison with Nos. 13 to 16. Nos. 13, 14, 15 and 16 are the safe pressures for struts of medium steel (ultimate strength about 70,000 pounds per square inch) for fixed, flat, pin and round ends respectively.

The factor of safety employed by the American Bridge Co. in calculating the safe pressures is as follows:—

$3 + {\cdot}01\ l/r$ for flat and fixed ends.

$3 + {\cdot}015\ l/r$ for hinged and round ends.

N.B.—For $l/r = 100$, the factors of safety are 4 and $4\frac{1}{2}$ respectively; for $l/r = 200$, they become 5 and 6 respectively.

In the case of round and square columns, the above allowances may be increased by 10 % and 5 % respectively (see Handbook, 1900, page 174).

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* At the time of going to press (June, 1906) a new handbook has been received from Messrs. Dorman, Long & Co. showing a new and reduced series of calculated destructive pressures. The factor of safety adopted is 4. The corresponding safe pressures are given at the foot of the table on page 166 (Reference No. 22).

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The American Bridge Co. define the terms relating to the end-fixing as follows :—

"In the class of 'fixed ends,' the struts are supposed to be so rigidly attached at both ends to the contiguous parts of the structure that the attachment would not be severed if the member was subjected to the ultimate load. 'Flat-ended' struts are supposed to have their ends flat and normal to the axis of length, but not rigidly attached to the adjoining parts. 'Hinged ends' embrace the class which have both ends properly fitted with pins, or ball and socket joints of substantial dimensions as compared with the section of the strut. 'Round-ended' struts are those which have only central points of contact, such as balls or pins resting on flat plates, but still the centres of the balls or pins coincident with the proper axis of the strut." And elsewhere: "If the strut is hinged by any uncertain method, so that the centres of pins and axis of strut may not coincide, or the pins may be relatively small and loosely fitted, it is best in such cases to consider the strut as 'round-ended.'"

(17) This is a combination of the straight line formula for long columns, with a maximum unit stress for short columns, as employed by the Carnegie Steel Co. for calculating tables of safe loads on columns built-up from channels etc.

The formula is :—

$$P' = 17{,}100 - 57\ l/r \text{ pounds for } l/r = 90 \text{ or more.}$$
$$P' = 12{,}000 \text{ pounds for } l/r < 90.$$

(See Carnegie Handbook, 1903, page 125.)

(18) Swiss Government Formula :—

$$\text{Safe Pressure} = 5{\cdot}1\ (1 - 0{\cdot}00375\ l/r) \text{ for } l/r < 110.$$
$$= 35030\ r^2 \div l^2 \qquad \text{for } l/r > 110.$$

This is a rational combination of the straight line and Euler formulæ. In plotting the curve corresponding to the Euler formula (No. 1), this has to be intersected by a horizontal straight line at $l/r = 65$ approximately, the safe stress at this point being 6 tons per square inch. This sharp distinction exhibited by the curve and the straight line is not real, because steel is not perfectly homogeneous and the form of the section, if not a solid square or circle, has a weakening effect on the elastic resistance of the column. It is therefore good practice to cut off the sharp corner by a curve, or by an inclined straight line as in the Swiss Government formula.

(19) This is a modification of the constants in the Swiss Government formula as proposed in a previous edition.

$$P = 6\ (1 - 0{\cdot}0059\ l/r) \text{ for } l/r < 110.$$
$$= 25{,}260\ r^2/l^2 \qquad \text{for } l/r > 110.$$

From $l/r = 110$, the results coincide with those of Euler's formula (No. 1) above.

(20) (21) These are variations of the Rankine formula adopted by the American Bridge Co. (Handbook, 1900, pages 268 to 277) as the "Permissible Strains for Compression Members" in the "Pencoyd Specifications for Railroad Bridges."

The formulæ are as follows :—

$$\text{For Medium Steel : } P' = 17{,}000 \div \left\{1 + \frac{1}{11{,}000}\frac{l^2}{r^2}\right\} \text{pounds.}$$

$$\text{For Soft Steel : } P' = 15{,}000 \div \left\{1 + \frac{1}{13{,}500}\frac{l^2}{r^2}\right\} \text{pounds.}$$

Medium steel to have an ultimate strength of 60,000 to 70,000 pounds (26·8 to 31·3 tons) per square inch. Soft steel to have an ultimate strength of 50,000 to 60,000 pounds (22·3 to 26·8 tons) per square inch. It is stipulated that "no compression-member, however, shall have a length exceeding 100 times its least radius of gyration, excepting those for wind-bracing, which may have a length not exceeding 120 times."

(22) These represent one-fourth of the calculated "Crippling Loads" as published by Messrs. Dorman, Long & Co. Limited, of Middlesbrough ("Pocket Companion," 1906, page 69). See also note to Formula No. 8 above.

ECCENTRIC LOADS ON STANCHIONS.

The formulæ by which the tabulated "coefficients" were calculated are given on page 174.

INTERIOR STANCHIONS.

When a stanchion carries floor girders upon brackets on opposite sides, it is generally supposed that the loading is symmetrical and that no bending-moment is caused, but it is obvious that the actual loads on the girders will vary as the distribution of the superimposed or live load is varied. In some cases, moreover, the girder on one side carries a larger floor area than the other, in which case it is clear that the unbalanced portion of its load is eccentric and exerts a corresponding pull on the stanchion.

INTERIOR STANCHION CARRYING EQUALLY LOADED GIRDERS ON OPPOSITE SIDES.

The ordinary table of safe loads on page 156 can be safely applied to all interior stanchions composed of Broad Flange Beams, provided that the estimated loads carried on opposite sides of the stanchion are equal or approximately so.* This remark applies equally, of course, to a stanchion loaded on all four sides. It is not necessary that all four loads should be equal, but each load must be balanced by the load on the opposite side.

INTERIOR STANCHION UNEQUALLY LOADED.

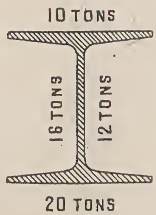
If the *estimated* loads on the girders are *not* equal, some additional margin of strength may be necessary.

The following example will show the mode of procedure in such cases :—

Suppose that four girders are connected to a stanchion 14 feet high and that these girders are estimated to carry maximum distributed loads as follows :—

Girders connected to flanges : 20 tons and 40 tons respectively.
,, ,, web : 24 tons and 32 tons ,,

Then the end-reactions, *i.e.* the loads transmitted to the stanchion, will be as follows :—



To make the example complete, assume that the stanchion also carries another stanchion above, transmitting a central load of 40 tons.

Then the loads may be set out as follows :—

(1) The stanchion carries central or balanced loads of $40+20+24=84$ tons.
An unbalanced load on one flange of .. .. .. 10 tons.
And an unbalanced load on one side of the web of .. 4 tons.

(2) Next, multiply the unbalanced portions of the load by $2\frac{1}{2}$ for flange connections and $1\frac{1}{2}$ for web connections, as a means of ascertaining approximately what section is required.†

* It is obvious that if the load on one side of a stanchion be removed so as to make the loading eccentric, the total load is necessarily reduced. In the case of stanchions composed of Broad Flange Beams, investigation shows that the reduction in the total load would always sufficiently compensate for the eccentricity. But this does not hold good for all types of stanchions.

† The same approximate coefficients, viz. $2\frac{1}{2}$ and $1\frac{1}{2}$ respectively, may be used in all cases as a means of arriving at an approximate size.

Then the load on the stanchion is found to be approximately equivalent to a central load of 115 tons as follows :—

Central or balanced loads	84 tons.
The unbalanced load on one flange is equivalent to 10 tons $\times 2\frac{1}{2}$ =	25 tons.
,, ,, ,, the web ,, ,, 4 tons $\times 1\frac{1}{2}$ =	6 tons.
∴ Total actual load on stanchion is equivalent to a central load of about	115 tons.

(3) Then, reference to the Table of Safe Loads (page 157) shows that section $12\frac{1}{2}'' \times 12''$ (safe load 114 tons) is near enough. As, however, the approximate coefficients used above are rather larger than those given in the table, a second calculation may be made to see whether a smaller size (12″ say) would be adequate. The coefficients of this size are given in the table as 2·36 and 1·19 respectively.

Consequently, if section $12'' \times 12''$ were used, the loads would be as follows :—

Central or balanced loads	84 tons.
The unbalanced load on one flange is equivalent to 10 tons $\times$ 2·36 =	23·6 tons.
,, ,, ,, the web ,, ,, 4 tons $\times$ 1·19 =	4·8 tons.
∴ Total actual load on stanchion (of 12″ section) is equivalent to approximately	112 tons.

However, on again referring to the table on page 157, the safe load for $12'' \times 12''$ section is found to be only 108 tons.

STANCHION CARRYING A GIRDER ON ONE SIDE ONLY.

This is the simplest and most frequently occurring case of serious eccentricity of loading. It is doubly serious if the girder is merely supported on a narrow bracket, as is commonly the case in ordinary building construction. Assuming, however, that the stanchion is of a wide-flanged section and that the fastenings are of the character illustrated in this book, the procedure will be as follows :—

(1) Suppose that the height of the stanchion is 18 feet and that the load is 50 tons. Then, if the girder is connected to one *flange* of the stanchion, a size must be used which will carry a central load of *about* 50 tons $\times 2\frac{1}{2}$ = 125 tons.

Turning to the ordinary table of safe loads on page 157, it will be found that either section 16″ (safe load 129 tons) or 15″ (safe load 121 tons) ought to be suitable.

By using the correct coefficient for section 15″, namely 2·39 (instead of the approximate coefficient $2\frac{1}{2}$), the eccentric load is found to be equivalent to a central load of $50 \times 2{\cdot}39 = 119$ tons, so that $15'' \times 12''$ is of adequate strength.

(2) If the stanchion is built into a brick wall with the flange outside, or otherwise braced laterally in such a way that it is secured against failure in the weaker direction, the special table of safe loads on page 162 should be used. By repeating the above process, using this special table, it will be found that $12\frac{1}{2}'' \times 12''$ is suitable.

(3) If the girder is connected to the *web* of the stanchion, the procedure is precisely similar, except, of course, that the coefficients for a web connection are used.

(4) A more convenient rule for proportioning external or corner stanchions is to make them of the same section as the inner stanchions. This is not an entirely safe rule for general use as, in many cases, the reduction of 50 % from the safe central load which it implies, would be an insufficient provision. But it may be quite safely followed in the case of stanchions composed of Broad Flange Beams, provided that the height of the stanchion is not more than about 20 times the flange width. The factor of safety is reduced from 5 to about 4·2, which is not objectionable under the circumstances.

[For Formulæ, see page 174.]

(5) The convenience of the foregoing rule is that it often simplifies the construction to have the whole of the stanchions on each floor of the same size and connected to the girders in the same fashion throughout. In cases where this is not desired, the girders should be attached to the webs of the outer stanchions whenever there is sufficient room between the flanges for the purpose, as in that event a lighter section can be used.

CORNER STANCHION CARRYING TWO GIRDERS AT RIGHT ANGLES.

Often a corner stanchion will carry two girders at right angles, one girder on the web and another girder on one of the flanges. In that case it will be proportioned in precisely the same way as explained for interior stanchions unequally loaded. Namely :—multiply each load by the suitable "coefficient for eccentric loading," taking $2\frac{1}{2}$ as the coefficient for the flange connection and $1\frac{1}{2}$ for the web connection in order to arrive at an approximate size. Then try if a smaller section will do when the loads are multiplied by the exact coefficients as given in the table for the smaller section in question.

FIXED ENDS ASSUMED.

It has been assumed throughout that the ends of the stanchions are fixed in the practical sense of the term. This is probably the case with regard to interior stanchions, but it is not always permissible to regard side and corner stanchions as having "fixed ends." Assuming, however, that the general construction is of the type recommended in this book, and that the side and corner stanchions form an integral part of the structure as in the case of a properly designed steel-framed building, then the ends of the stanchions may be safely regarded as fixed to the extent assumed in the ordinary table of safe loads on page 156. The same assumption may be made if the stanchion is built into or securely anchored to an outer wall of adequate thickness.

If the girders are simply bolted to a bracket as in the case of cast-iron or round columns, it may not be assumed that the ends of the stanchion are fixed, and the stanchions may have to be proportioned to a load of from 4 to 8 times the actual load.

STANCHIONS HAVING ECCENTRIC BASES OR FOUNDATIONS.

A stanchion cannot be regarded as centrally loaded, even if the load is applied directly over its axis, unless the foundation is symmetrical and the baseplate projects equally on opposite sides. When, owing to the proximity of a party wall, or from any other cause, the baseplate, or the foundation below, projects more on one side than the other, bending stress will be induced in the stanchion. If the stanchion has been calculated for an eccentric load as in the above examples, the baseplate should have the same amount of eccentricity (*i.e.* the centre of the baseplate should be directly under the load) ; but if the stanchion has been designed for a central load and it is desired to make the baseplate eccentric, a larger section of stanchion will be required just as if the loading were eccentric, and the baseplate should have its width extended to reduce the intensity of the stress on the edge nearest the stanchion.

FORMULÆ FOR CALCULATING COEFFICIENTS FOR ECCENTRIC LOADING ON STANCHIONS.

The undernoted formulæ are derived, on the advice of Professor Adams (M.Inst.C.E., M.I.Mech.E. etc.), from the principle that the sum of the stresses due to the vertical load and to the bending-moment should not exceed the stress allowed for central loading.

That is:—If P = Safe pressure for a central load (tons per square inch).
W' = Safe eccentric load (tons).
A = Sectional area (square inches).
M = Bending-moment due to the eccentricity of the load (ton-inches).
Z = Section Modulus (inches).

$$\text{Then } P=\frac{W'}{A}+\frac{M}{Z} \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (1)$$

The bending-moment (M) is measured in ton-inches by the product $W \times E$, where W = actual load (tons) and E = distance (inches) from the centre of the stanchion to the point at which the load is applied. In calculating the tabulated coefficients for eccentric loading, E has been taken as equal to one-half of the depth of the section in the case of a flange connection and one-half of the web thickness for a web connection. [This is correct if the connections are of the character illustrated in this book. But, if the load on the stanchion be transmitted by a girder poorly connected to it, *e.g.* merely carried on a bracket, the distance E should obviously be measured from the centre of the stanchion to the centre of the bearing surface on the bracket. This will greatly increase the bending-moment, especially if the connection is to the web of the stanchion.]

Let H = depth of beam (inches) and T = web thickness (inches).

Let W_x and W_y = Safe eccentric loads (tons) for flange and web connections respectively.

Let M_x and M_y = Respective bending-moments (ton-inches) corresponding to above loads.

$$\text{Then } M_x=W_x \times \tfrac{1}{2}\,H \text{ and } M_y=W_y \times \tfrac{1}{2}\,T \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (2)$$

Let Z_x and Z_y = Greatest and Least Section Moduli (inches) respectively.

Let C_x and C_y = Coefficients as given in table for flange and web connections respectively. P[illegible] 156

Let W = Safe centrally applied load (tons).

Then, substituting the above symbols in equation (1), we have

$$W_x=A\left[\frac{W}{A}-\frac{\frac{1}{2}H.W_x}{Z_x}\right] \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (3)$$

$$\text{and } W_y=A\left[\frac{W}{A}-\frac{\frac{1}{2}TW_y}{Z_y}\right] \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (4)$$

Equations (3) and (4) can be written:—

$$W=W_x \times C_x \text{ and } W=W_y \times C_y$$

$$\text{where } C_x=A\left\{\frac{1}{A}+\frac{\frac{1}{2}H}{Z_x}\right\}=\text{Constant for any given section of strut,}$$

$$\text{and } \quad C_y=A\left\{\frac{1}{A}+\frac{\frac{1}{2}T}{Z_y}\right\}=\text{Constant for any given section of strut.}$$

The tabulated coefficients for eccentric loading were calculated by these formulæ.

BROAD FLANGE BEAMS AS STRUTS.

(1) Values of l/r for various heights or lengths from 8 to 40 feet.
(2) Safe loads (tons) corresponding to working stresses from 1 to $7\frac{1}{2}$ tons per square inch.

Section.	r	Values of $l \div r$ for various heights from 8 to 40 feet. r=least radius of gyration. l=height in inches.																		
		8	9	10	11	12	13	14	15	16	17	18	19	20	22	24	28	32	36	40
Inches.	Ins.																			
7×7	1·67	57	65	72	79	86	93	100	108	115	122	129	136	144	158	173	201	230	258	287
8×8	1·86	52	58	65	71	78	84	90	97	103	110	116	123	129	142	155	181	207	233	258
$8\frac{1}{2}$×$8\frac{1}{2}$	2·04	47	53	59	65	71	77	82	88	94	100	106	112	118	129	141	165	189	212	235
$9\frac{1}{2}$×$9\frac{1}{2}$	2·21	44	49	54	59	65	71	76	82	87	93	98	103	109	120	131	153	174	196	218
10×10	2·30	42	47	52	57	63	68	73	79	84	89	94	100	105	115	126	147	168	189	209
$10\frac{1}{4}$×$10\frac{1}{4}$	2·39	40	45	50	55	60	65	70	75	80	85	90	95	100	110	120	140	160	180	201
$10\frac{1}{2}$×$10\frac{1}{2}$	2·49	39	43	48	53	58	63	68	72	77	82	87	92	97	106	116	135	154	174	193
11×11	2·58	37	42	46	51	56	61	65	70	74	79	84	88	93	102	112	130	149	168	186
$11\frac{1}{2}$×$11\frac{1}{2}$	2·66	36	41	45	50	54	59	63	68	72	77	81	86	90	99	108	126	145	163	181
12×12	2·76	35	39	43	48	52	57	61	65	70	74	78	83	87	96	104	122	139	157	174
$12\frac{1}{2}$×12	2·75	35	39	44	48	52	57	61	66	70	74	79	83	87	96	105	122	139	157	175
$13\frac{1}{2}$×12	2·74	35	40	44	48	53	57	61	66	70	75	79	84	88	97	105	123	141	158	175
14×12	2·74	35	40	44	48	53	57	61	66	70	75	79	84	88	97	105	123	141	158	175
15×12	2·73	35	40	44	49	53	57	62	66	71	75	79	84	88	97	106	124	141	159	176
16×12	2·72	35	40	44	49	53	57	62	66	71	75	80	84	88	97	106	124	142	159	177
17×12	2·70	36	40	44	49	53	58	62	67	71	76	80	85	89	98	107	125	143	160	178
18×12	2·68	36	40	45	49	54	58	63	67	72	76	81	85	90	99	107	125	143	161	179
19×12	2·67	36	41	45	50	54	59	63	68	72	77	81	86	90	99	108	126	144	162	180
20×12	2·63	36	41	46	50	55	59	64	69	73	78	82	87	91	101	110	128	146	165	183
22×12	2·60	37	42	46	51	55	60	65	69	74	79	83	88	92	102	111	129	148	166	185
24×12	2·56	38	42	47	52	56	61	66	70	75	80	84	89	94	103	113	131	150	169	188
26×12	2·51	38	43	48	53	57	62	67	72	77	81	86	91	96	105	115	134	153	172	191
30×12	2·43	40	45	49	54	59	64	69	74	79	84	89	94	99	109	119	138	158	178	198

Section.	A	P×A, *i.e.* Safe Loads in tons corresponding to various working stresses. A=Sectional Area (square inches). P=Working Stress (tons per square inch).													
		1	$1\frac{1}{2}$	2	$2\frac{1}{2}$	3	$3\frac{1}{2}$	4	$4\frac{1}{2}$	5	$5\frac{1}{2}$	6	$6\frac{1}{2}$	7	$7\frac{1}{2}$
Inches.	sq. ins.														
7×7	9·3	9	14	19	23	28	33	37	42	46	51	56	60	65	70
8×8	10·9	11	16	22	27	33	38	44	49	55	60	65	71	76	82
$8\frac{1}{2}$×$8\frac{1}{2}$	12·8	13	19	26	32	38	45	51	58	64	70	77	83	90	96
$9\frac{1}{2}$×$9\frac{1}{2}$	15·0	15	23	30	38	45	53	60	68	75	83	90	98	105	113
10×10	16·3	16	24	33	41	49	57	65	73	81	90	98	106	114	122
$10\frac{1}{4}$×$10\frac{1}{4}$	17·9	18	27	36	45	54	63	72	81	90	99	108	116	125	134
$10\frac{1}{2}$×$10\frac{1}{2}$	19·1	19	29	38	48	57	67	76	86	96	105	115	124	134	143
11×11	20·4	20	31	41	51	61	72	82	92	102	112	123	133	143	153
$11\frac{1}{2}$×$11\frac{1}{2}$	21·9	22	33	44	55	66	77	87	98	109	120	131	142	153	164
12×12	23·6	24	35	47	59	71	83	94	106	118	130	141	153	165	177
$12\frac{1}{2}$×12	24·9	25	37	50	62	75	87	100	112	125	137	149	162	174	187
$13\frac{1}{2}$×12	25·9	26	39	52	65	78	91	104	117	130	143	156	169	182	195
14×12	28·2	28	42	56	70	84	99	113	127	141	155	169	183	197	211
15×12	29·6	30	44	59	74	89	104	119	133	148	163	178	193	207	222
16×12	31·6	32	47	63	79	95	110	126	142	158	174	189	205	221	237
17×12	33·2	33	50	66	83	99	116	133	149	166	182	199	216	232	249
18×12	35·6	36	53	71	89	107	124	142	160	178	196	213	231	249	267
19×12	37·5	38	56	75	94	113	131	150	169	188	206	225	244	263	281
20×12	40·6	41	61	81	101	122	142	162	183	203	223	243	264	284	304
22×12	44·7	45	67	89	112	134	156	179	201	223	246	268	290	313	335
24×12	46·6	47	70	93	116	140	163	186	210	233	256	280	303	326	349
26×12	48·8	49	73	98	122	146	171	195	219	244	268	293	317	341	366
30×12	52·0	52	78	104	130	156	182	208	234	260	286	312	338	364	390

EXPLANATION OF THE ABOVE TABLES.

The above tables will be found useful in making special calculations of the strength of Broad Flange Beams as struts etc. The first table gives the values of l/r for a series of heights corresponding to those given in the tables of safe loads. The second table gives the values of P×A for various values of P from 1 to $7\frac{1}{2}$ tons per square inch. P denotes a given working stress or safe pressure per square inch, A denotes the sectional area of the strut, and consequently, the tabulated values of P×A denote the safe *loads* on Broad Flange Struts corresponding to the various given working stresses.

Foundations.
Misc. Shapes.
Misc. Tables.
Prices etc.
Code.
Grey Mill.
Tests etc.
R.I.B.A.
Photos.

FOUNDATIONS FOR STEEL STANCHIONS.

The foundations for steel stanchions require particular care from the fact that the position cannot be selected but is dependent upon the general design of the building. Although the average character of the soil may be known, and the foundation designed in accordance therewith, it is necessary to examine the excavation for each individual stanchion so that the foundations may be modified if the circumstances indicate that it would be desirable. This is more especially the case with regard to the depth of concrete. It is necessary to reach a hard bottom, and this is often found at different depths within the space of a few yards; for instance, in putting in the foundations for a large coal wharf alongside the river Thames, the estimated depth before a hard stratum would be reached was 16 feet, whereas in the excavation of the work it was found to vary from 15 feet to 17 feet 6 inches, and the concrete was put in accordingly. When the soil is uniform for a considerable depth it is only necessary to make the concrete deep enough to prevent the shearing of the projecting edges, and this will be attained when the depth is not less than $1\frac{1}{2}$ times the projection beyond the material next above. The bearing power of soils is variously given by different authorities as from 2 to 8 tons per square foot for gravel, deep hard clay and natural compact earth, although seldom more than 5 tons is put upon the very best of them; while made ground is estimated to carry only from $\frac{1}{2}$ to 1 ton per foot super. The weight of the foundation itself should be taken into account as well as that of the stanchion and the load upon it, in estimating the pressure upon the soil.

Foundations for stanchions should be absolutely rigid, the slightest settlement may alter the distribution of stress in the girder work with possibly disastrous results; on the other hand, any surplus material in a foundation beyond that actually required for stability is so much buried money. If the baseplate of a stanchion have a sufficient area it may rest direct upon Portland cement concrete, but usually other materials are interposed so that a small area of baseplate is gradually extended in each successive material until the intensity of pressure is reduced to that which can be safely resisted by the soil. In America, where heavy loads are found in conjunction with poor soils, it is common to put a cast-iron bearing block under the stanchions, and below this successive layers of rolled steel joists bedded in cement concrete in alternate directions until a sufficient spread is obtained. In Great Britain it is more usual to bed the stanchions on blocks of ashlar stone, with brick pier and footings below, and cement concrete underneath. Whichever method is adopted it is necessary to consider the bearing power of the lower material in deciding the area of that immediately above.

The published figures of the strength of building materials should be used with great caution, or overloading will be the result; the figures are based upon actual tests, but the circumstances are totally different from those arising in practice. When a sample of stone is to be tested, it is not only specially selected but is carefully bedded between pieces of pine, so that the pressure shall be distributed as uniformly as possible; and for a similar reason a sample of brick is brought to a true face by a coating of plaster of Paris. Without particularising the conditions that are found in practice, it is enough to say that even in the most careful work they do not approach those stated above, and there are various other contingencies to be allowed for, so that a factor of safety only nominally represents the actual margin between the working stress and the ultimate limit in each case. The compressive strength of concrete depends not only upon its composition but also to a very great extent upon the care with which it is mixed; it is easy to lose half the strength

by careless handling. The following table gives approximately the ultimate and safe loads upon ordinary good concrete in tons per foot super at the end of twelve months.

Composition.	Crushing Load.	Fracture Load, say	Safe Load Factor, about 2	4
1 P.C. to 1 ballast	150 tons	100 tons	50 tons	25 tons
,, 2 ,,	118 ,,	78 ,,	39 ,,	20 ,,
,, 3 ,,	98 ,,	65 ,,	32½ ,,	16 ,,
,, 4 ,,	86 ,,	57 ,,	28½ ,,	14 ,,
,, 5 ,,	76 ,,	50 ,,	25 ,,	12½ ,,
,, 6 ,,	67 ,,	45 ,,	22½ ,,	11 ,,
,, 7 ,,	59 ,,	40 ,,	20 ,,	10 ,,
,, 8 ,,	52 ,,	35 ,,	17½ ,,	9 ,,
,, 9 ,,	46 ,,	30 ,,	15 ,,	7½ ,,
,, 10 ,,	42 ,,	28 ,,	14 ,,	7 ,,

Say 35 % less at 1 month, 15 % less at 3 months, 7½ % less at 6 months, and 2½ % less at 9 months, than given in above table.

The Austrian Association of Engineers and Architects found the ultimate strength of concrete 1 to 5 to be 114 tons per square foot, 1 to 8 to be 59 tons, and 1 to 10 to be 46 tons.

Prof. Turneaure was able to make 3 inch cubes of 1 cement 2 sand and 4 sized aggregate up to ¾ inch to stand 128 tons per square foot at 1 month and 160 tons per square foot at 3 months; while the Trussed Concrete Steel Co. gave 3,000 lbs. per square inch=192·8 tons per square foot as the ultimate strength they had reached; but the New York Building Regulations fix the safe load at 350 lbs. per square inch=22·5 tons per square foot without specifying the mixture to be employed.

In British practice the table given above is a reasonable one to adopt when the loads are fairly estimated.

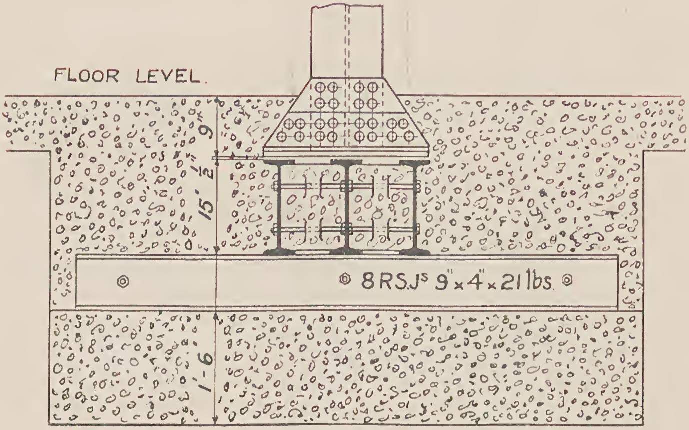
The other materials used in foundations may be estimated as follows per square foot:—

Material.	Crushing Load.	Fracture Load, say	Safe Load, say
Granite	900 tons	300 tons	30 tons
Portland and compact limestone	500 ,,	200 ,,	20 ,,
Hard York	400 ,,	120 ,,	15 ,,
Ordinary limestone ...	120 ,,	60 ,,	6 ,,
Blue bricks in cement ...	180 ,,	72 ,,	12 ,,
Stock ,, ,, ...	120 ,,	30 ,,	8 ,,
,, ,, lias mortar ...	50 ,,	20 ,,	6 ,,
,, ,, grey lime mortar	16 ,,	12 ,,	4 ,,

All the figures for safe loads given in the above notes may be considered as the maximum under favourable circumstances; due allowance should be made if the work is not under competent and continuous supervision.

[For Table, see page 183.]

Fig. 1



GRILLAGE FOUNDATIONS.

Grillage foundations are a little troublesome to design without previous experience, but the principles of construction are simple :—

(1) Area of Foundation. The safe bearing capacity of the soil must be determined in the usual manner. Suppose that this is found to be 3 tons per square foot and that the total load on the foundation is 108 tons, then, obviously, the area of the foundation should be 108÷3=36 square feet. Hence, a foundation 6 feet square would be suitable.

(2) Number of Layers of Joists. This depends on the size of the foundation and of the stanchion base. For exceptionally heavy loads, three or more layers of joists may be required. For light stanchions, as for walls, a single layer of joists may be sufficient.

(3) Thickness of Concrete under and round Joists. The layer of concrete under the joists is generally made 12 to 18 inches thick. To prevent corrosion, it is necessary that every portion of the steel beams be protected by a covering of good Portland cement concrete, at least 3 inches thick.

For this reason it is desirable to leave a space of 3 to 6 inches or more between the flanges of the joists in each layer, so as to enable the concrete to be properly rammed between the joists.

(4) Shape of Foundation. This is usually square. But when only one layer of joists is used, the foundation will be rectangular in shape.

(5) Number and Spacing of Joists. *(a)* The *Bottom Layer* of joists should be spaced about 12 to 18 inches apart, centre to centre. The joists may be placed somewhat closer if desired, but the clear distance between the edges of the flanges of the joists should not be less than 3 inches in any case, as otherwise it would be difficult or impossible to fill in properly with concrete. On the other hand, if the distance centre to centre exceeds about 18 inches, the pressure on the concrete under the joists is likely to be excessive.

(b) The *Top Layer*, if the stanchion is a single Broad Flange Beam, will consist of three joists arranged as in Figs. 1 and 3, or it is possible to use four joists with 5" flanges if preferred. In other cases, the joists will

Figs. 1 to 3. The illustrations show a stanchion consisting of Broad Flange Beam section 19"×12" carrying a load of 180 tons (height 11 feet). The foundation is 8 feet square, thus reducing the pressure per square foot on the substratum (dry clay) to about 3 tons per square foot.

[For Table, see page 183.]

Fig. 2

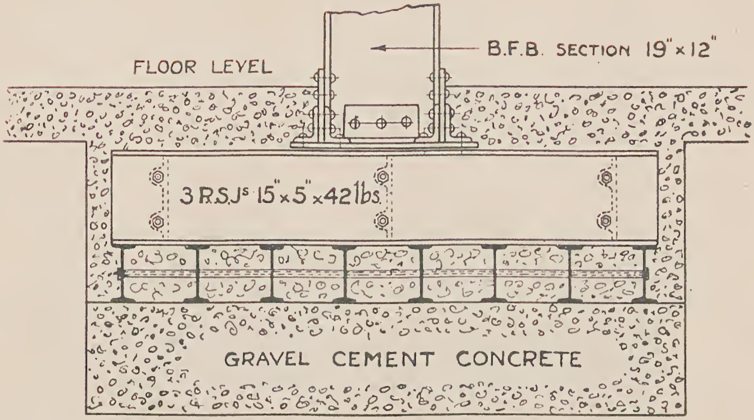
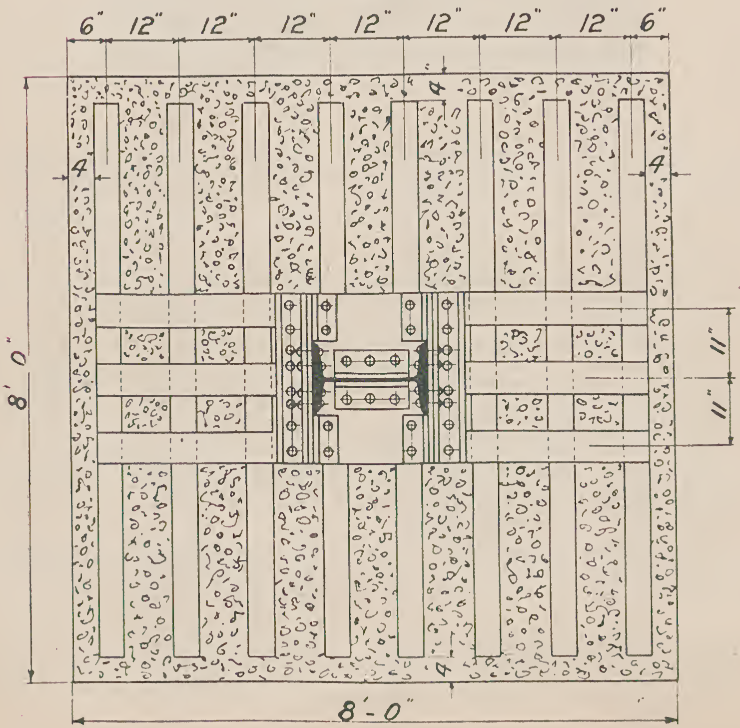


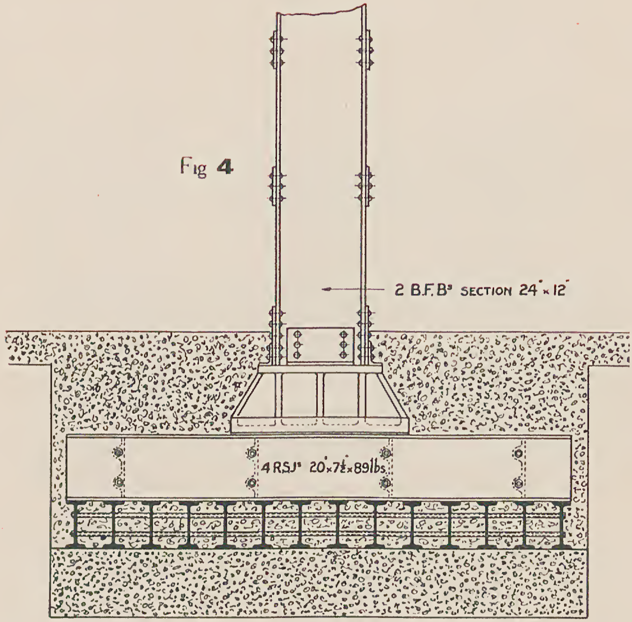
Fig. 3



Misc. Shapes.
Misc. Tables.
Prices etc.
Code.
Grey Mill.
Tests etc.
R.I.B.A.
Photos.

[For Table, P.T.O.]

Fig 4



usually have to be spaced as closely as practicable, say 9 to 10 inches apart, centre to centre, according to the flange width of the joists. This is necessary in order to provide adequate web area to resist shear and buckling. (See also following paragraph.)

(c) There is no difficulty in determining the correct spacing of the joists. On making a sketch plan, it is fairly obvious how many joists can be used in each layer, and this will decide the spacing.

(6) Separators. When two or more layers of joists are used, the joists in the bottom layer are usually tied together by bolts passing through pipe "separators" consisting of steel tube, say 1" diameter and of metal ⅛" or $\frac{3}{16}$" thick. (See Figs. 2 and 4.)

The joists in the upper layer are generally held in position by cast-iron "separators" as in Figs. 1 and 5. Sometimes short lengths of steel channels are used instead. The former should be ground to fit the flanges of the joists. Their function is to support the webs of the joists, these being usually of a relatively deep section, and to resist any unsetting tendency.

Grillage joists are sometimes placed so close together as to render it impossible to ram the concrete between the flanges. In such cases tube separators or some other type of construction will have to be adopted, which permits the spaces between the joists to be completely filled in from the ends. Consequently, the resistance of the webs to buckling or shear will have to be considered with especial care.

(7) It is important to provide a rigid base of substantial area. Failing this, there is a danger of the webs of the grillage joists buckling under the load. For heavy stanchions, as illustrated in Fig. 4, cast-iron bases are preferable to steel bases.

Figs. 4 to 6. The illustrations show a grillage foundation designed for a load of 450 tons. The stanchion consists of two Broad Flange Beams of 24" × 12" section, tied as shown at 3 feet centres. The foundation is 12 feet square so as to reduce the pressure per square foot on the substratum (dry clay) to about 3 tons per square foot.

[For Table, P.T.O.]

Fig. 5

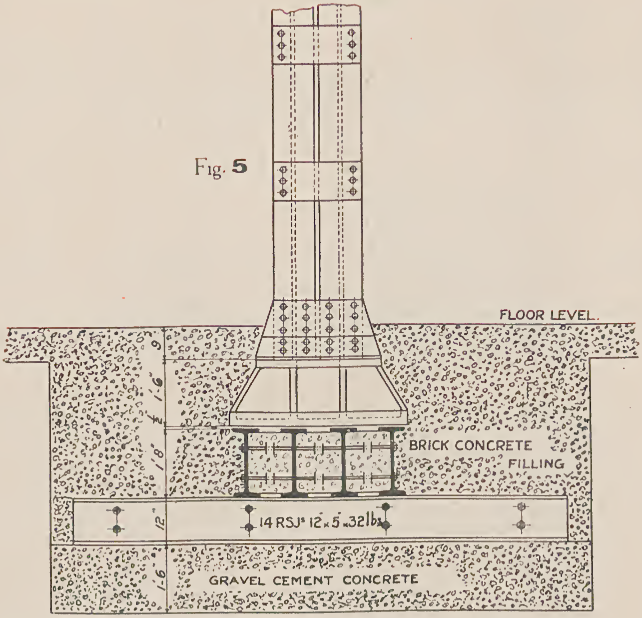
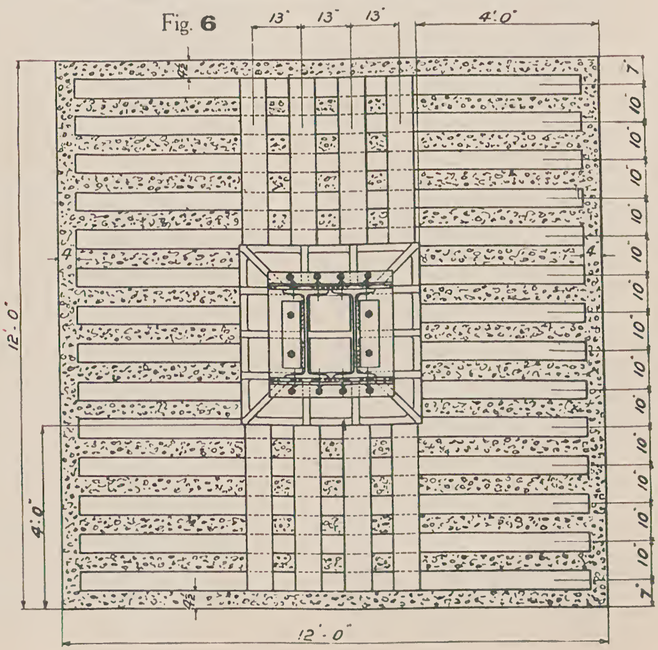


Fig. 6

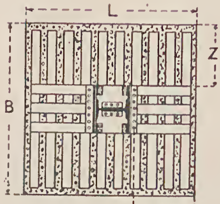


Misc. Shapes.
Misc. Tables.
Prices etc.
Code.
Grey Mill.
Tests etc.
R.I.B.A.
Photos.

SIZE OF JOISTS.

(*a*) The principles on which grillage joists are proportioned are that the joists are assumed to be firmly fixed under the base and uniformly loaded from the underside. Accordingly, the free or projecting lengths are treated as cantilevers. The weight of the foundation itself is neglected.

Fig. 7.



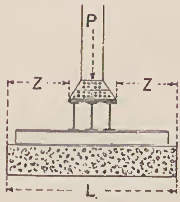


Fig. 8.

(*b*) Let P=Total Load on Foundation (tons), *i.e.* Load transmitted by Stanchion.

Let L and B=Length and Breadth of Foundation (inches).

Let Y and Z=Free or projecting lengths of the Grillage Joists (inches).

N.B.—It will be observed from Fig. 7 that the projecting length "Y" is measured from the centre line of the rivets. If the stanchion were fitted with a cast-iron base, the length "Y" would be measured from the outer edge of the base.

(*c*) Taking the upper layer of joists in Fig. 7:—It follows from § (*a*) that:—

Maximum Bending-moment=$P \times Y^2 \div 2.L$ ton-inches.

That is:—To find the maximum bending-moment, multiply the total load on the foundation (P tons) by the square of the projection Y (inches) and divide by twice the length L (inches). *E.g.* Applying this formula to Fig. 4, the result is found to be 3,600 ton-inches.

(*d*) The number of joists in the layer will already have been determined in accordance with § 5 on page 178 ("Number and Spacing of Joists"); they must have a total Moment of Resistance equal to the ascertained bending-moment. *E.g.* Reverting to Fig. 4, it will be observed that 4 joists are used, and the maximum bending-moment has already been ascertained to be 3,600 ton-inches. Now refer to the column headed "4 R" in the table opposite ("4" denotes the number of joists, "R" their combined Moment of *Resistance*). It will be observed that the smallest joist section of the required strength is 18″ × 7″.

(*e*) Next ascertain whether the shearing stress in the webs of the beams is excessive. *E.g.* In the column headed "4 S" opposite 18″ × 7″ the maximum permissible load is given as 356 tons. But the load on the stanchion in Fig. 4 is 450 tons. Consequently, a heavier size must be used. 20″ × 7½″ is also slightly below the required strength, but the difference in this case may be permitted.

N.B.—In the table opposite, the permissible shearing stress is taken as 4½ tons per square inch. Theoretically, the shear is $P \times Y \div L$; but in order to be on the safe side, it is better to assume the shear to be equal to one-half of the total load, as in the table.

(*f*) The requisite size of the joists in the bottom layer is determined in precisely similar fashion. The maximum bending-moment will be $(P \times Z^2) \div 2B$.

E.g. In Fig. 4, the bottom layer consists of 14 joists which are required to have a combined moment of resistance of 3,600 ton-inches. As the table opposite only goes up to 12 joists, look in the column headed "7" for a section suited to a bending-moment (Column "R") of 1,800 ton-inches (3,600÷2=1,800) and to a total load (Column "S") of 225 tons (450÷2=225). Section 12″ × 5″ is seen to be the smallest size of the required strength.

N.B.—In Fig. 4, as in all other cases where the foundation comprises two layers of joists and the stanchion base and foundation are square in shape, the bending-moment is obviously the same for both layers of joists.

GRILLAGE JOISTS.—Continued.

TABLE FOR SELECTING GRILLAGE JOISTS.

Size. A × B	Weight per foot.	Section Modulus. XX	Web Thickness T	Web Area. T × A	3 R	3 S	4 R	4 S	5 R	5 S	6 R	6 S
Inches.	Lbs.	Ins.³	Ins.	Sq. Ins.	Ton-inches.	Tons.	Ton-inches.	Tons.	Ton-inches.	Tons.	Ton-inches.	Tons.
9 × 4	21	18·0	·300	2·70	406	73	541	97	676	122	811	146
10 × 5	30	29·1	·360	3·60	655	97	873	130	1092	162	1311	194
12 × 5	32	36·7	·350	4·20	826	113	1101	151	1376	189	1651	227
10 × 6	42	42·3	·400	4·00	952	108	1270	144	1587	180	1904	216
9 × 7	58	51·1	·550	4·95	1149	134	1532	178	1915	223	2297	267
12 × 6	44	52·6	·400	4·80	1183	130	1577	173	1972	216	2366	259
15 × 5	42	57·1	·420	6·30	1285	170	1713	227	2141	284	2569	340
12 × 6	54	62·6	·500	6·00	1409	162	1878	216	2348	270	2817	324
14 × 6	46	62·9	·400	5·60	1416	151	1888	202	2361	252	2833	302
14 × 6	57	76·2	·500	7·00	1714	189	2285	252	2856	315	3427	378
15 × 6	59	83·9	·500	7·50	1887	203	2516	270	3146	338	3775	405
16 × 6	62	90·7	·550	8·80	2042	238	2722	317	3403	396	4084	475
18 × 7	75	128	·550	9·90	2874	267	3832	356	4791	446	5749	535
20 × $7\frac{1}{2}$	89	167	·600	12·0	3761	324	5014	432	6268	540	7521	648

TABLE FOR SELECTING GRILLAGE JOISTS.—Continued.

Size. A × B	Weight per foot.	7 R	7 S	8 R	8 S	9 R	9 S	10 R	10 S	11 R	11 S	12 R	12 S
Inches.	Lbs.	Ton-inches.	Tons.	Ton-inches.	Tons.	Ton-inches.	Tons.	Ton-inches.	Tons.	Ton-inches.	Tons.	Ton-inches.	Tons.
9 × 4	21	946	170	1082	194	1217	219	1352	243	1487	267	1622	292
10 × 5	30	1529	227	1748	259	1966	292	2185	324	2404	356	2622	389
12 × 5	32	1926	265	2202	302	2477	340	2752	378	3027	416	3302	454
10 × 6	42	2222	252	2539	288	2857	324	3174	360	3491	396	3809	432
9 × 7	58	2680	312	3063	356	3446	401	3829	446	4212	490	4595	535
12 × 6	44	2760	302	3154	346	3549	389	3943	432	4337	475	4732	518
15 × 5	42	2997	397	3426	454	3854	510	4282	567	4710	624	5139	680
12 × 6	54	3287	378	3756	432	4226	486	4695	540	5165	594	5634	648
14 × 6	46	3305	353	3777	403	4249	454	4721	504	5193	554	5665	605
14 × 6	57	3998	441	4570	504	5141	567	5712	630	6283	693	6854	756
15 × 6	59	4404	473	5033	540	5662	608	6291	675	6920	743	7549	810
16 × 6	62	4764	554	5445	634	6125	713	6806	792	7487	871	8167	950
18 × 7	75	6707	624	7665	713	8623	802	9581	891	10539	980	11497	1069
20 × $7\frac{1}{2}$	89	8775	756	10028	864	11282	972	12535	1080	13789	1188	15042	1296

EXPLANATION OF THE ABOVE TABLE.

(1) The figures in the columns headed "R" are the total Moments of Resistance (ton-inches) of various numbers of joists (from 3 to 12). The working stress is taken as $7\frac{1}{2}$ tons per square inch. That is:—"R" = Section Modulus (xx) × $7\frac{1}{2}$ × Number of Joists.

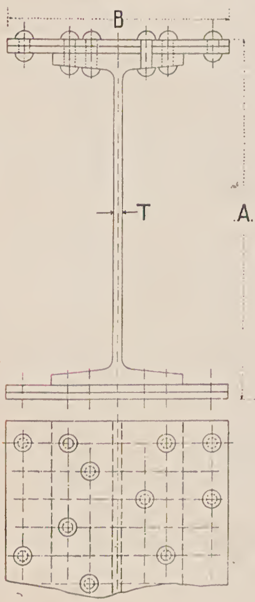
(2) The figures in the columns headed "S" represent the maximum permissible load (tons) on the whole foundation. The safe shearing stress is taken as $4\frac{1}{2}$ tons per square inch and the maximum shear on each set of joists is assumed to be equal to one-half of the total load on the foundation. That is:—"S" = 2 × Depth of Joist (A) × Web Thickness (T) × $4\frac{1}{2}$ × Number of Joists.

N.B.—The sections are tabulated in order of strength as determined by the Moments of Resistance.

BROAD FLANGE BEAMS WITH FLANGE PLATES.

LIST OF SIZES AND PROPERTIES.

Approximate Size.		Weight per foot.	Depth of plain beam.	Plates.			Web.	Sectional Area.	Moments of Inertia.		Section Moduli.		Radii of Gyration.		Reference No.
A	B			No.	Thick.	Width.	T		XX	YY	XX	YY	XX	YY	
Inches.		Lbs.	Ins.		Ins.	Ins.	Ins.	Sq. Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	
9½	× 10	80	8½	2	½	10	·35	21·2	357	125	74·0	24·9	4·11	2·43	1202
10½	× 12	94	9½	2	½	12	·39	25·4	510	199	97·1	33·1	4·48	2·81	1204
11	× 12	103	10	2	$\frac{9}{16}$	12	·41	28·0	622	222	112	37·1	4·76	2·83	1206
11½	× 12	114	10¼	2	⅝	12	·43	30·8	733	254	128	42·2	5·00	2·85	1208
12	× 12	123	10½	2	$\frac{11}{16}$	12	·44	33·2	856	281	143	46·8	5·08	2·91	1210
12½	× 14	143	11	2	¾	14	·45	38·9	1115	433	178	61·8	5·35	3·34	1212
13	× 14	148	11½	2	¾	14	·47	40·2	1226	448	189	64·0	5·52	3·34	1214
13½	× 14	154	12	2	¾	14	·49	41·9	1363	473	202	67·6	5·70	3·36	1216
14	× 14	158	12½	2	¾	14	·51	43·2	1539	481	220	67·7	5·97	3·34	1220
15	× 16	172	13½	2	¾	16	·53	47·2	1949	655	260	81·9	6·43	3·73	1224
16	× 16	193	14	4	$\frac{7}{16}$	16	·56	53·1	2419	751	307	93·8	6·51	3·76	1228
17	× 16	212	15	4	½	16	·58	57·8	3019	832	355	104	7·23	3·79	1232
18	× 18	232	16	4	½	18	·61	61·6	3529	1019	398	113	7·57	4·07	1236
19	× 18	254	17	4	$\frac{9}{16}$	18	·63	67·2	4493	1128	473	125	8·18	4·10	1240
20	× 18	262	18	4	$\frac{9}{16}$	18	·67	69·5	5006	1142	501	127	8·49	4·06	1248
21	× 18	268	19	4	$\frac{9}{16}$	18	·69	71·5	5669	1151	540	128	8·91	4·02	1256
22	× 20	294	20	4	$\frac{9}{16}$	20	·76	78·8	6891	1551	626	155	9·35	4·44	1264
24	× 20	308	22	4	$\frac{9}{16}$	20	·81	82·7	9653	1570	804	157	10·8	4·36	1272
28	× 20	339	26	4	⅝	20	·83	90·3	12858	1718	918	172	11·9	4·36	1280
32	× 20	350	30	4	⅝	20	·83	94·5	16935	1718	1058	172	13·4	4·27	1288



EXPLANATION OF THE ABOVE TABLE.

(1) The weights per foot include for rivet heads.

(2) In calculating the moments of inertia, sectional area etc., proper allowance has been made for rivet holes.

(3) The "safe distributed loads" are calculated for a working stress of 7½ tons per square inch.

(4) The safe loads on stanchions are calculated by the same formula as the table on page 156 ("Fidler's":—Factor of Safety 5).

(5) None of the tabular loads cause a calculated deflection exceeding $\frac{1}{25}$th of an inch per foot.

(6) In the case of loads printed in italics, stiffeners should be used over the supports. [See table of maximum loads on plain beams, page 140.]

(7) The flange plates need not extend to the full length of the girder.

(8) The third line of rivets shown in drawing is only required for 18″ and 20″ plates.

BROAD FLANGE BEAMS WITH FLANGE PLATES.—Continued.

SAFE LOADS AS GIRDERS AND STANCHIONS.

Depth of Section.	SAFE DISTRIBUTED LOADS AS GIRDERS (TONS).						SAFE LOADS AS STANCHIONS (TONS).						Reference No. and Code Word.	
	32′	36′	40′	44′	48′	52′	14′	16′	18′	20′	24′	30′		
Inches.														
9½	...	...	...	...	...	...	92	87	81	73	60	44	1202	Inebriate
10½	...	...	...	...	...	...	117	111	105	100	84	65	1204	Infamy
11	...	...	...	...	...	...	130	123	117	110	93	73	1206	Infant
11½	...	...	...	...	...	...	143	136	128	122	103	81	1208	Infection
12	...	...	...	...	...	...	154	147	140	134	114	89	1210	Inference
12½	...	...	...	...	...	...	187	182	174	166	152	122	1212	Infidel
13	...	...	...	...	...	...	193	188	180	172	157	126	1214	Infirmity
13½	...	...	...	...	...	...	201	196	189	180	163	133	1216	Influenza
14	34	30	27	...	...	...	207	202	193	185	169	136	1220	Influx
15	41	36	32	29	...	...	232	226	220	212	195	166	1224	Informer
16	48	43	38	35	32	...	261	254	248	239	220	189	1228	Infusion
17	55	49	44	40	37	...	286	276	270	262	240	209	1232	Ingenuity
18	62	55	50	45	41	...	309	300	292	286	265	237	1236	Ingrate
19	74	66	59	54	49	45	337	327	319	312	291	259	1240	Ingress
20	78	70	63	57	52	48	349	339	330	323	299	266	1248	Injection
21	84	75	67	61	56	52	357	346	338	330	305	271	1256	Inkstand
22	98	87	78	71	65	60	400	392	380	372	352	319	1264	Inlay
24	126	112	101	91	84	77	419	409	397	389	367	330	1272	Inlet
28	*144*	128	115	104	96	88	457	446	433	424	401	360	1280	Inmate
32	*165*	*147*	*132*	120	110	102	478	464	452	442	417	374	1288	Insanity

FOR EXPLANATION OF ABOVE TABLE, SEE OPPOSITE PAGE.

USES OF PLATED BROAD FLANGE BEAMS.

It is needless to say that plain Broad Flange Beams should be used if possible, as "plating" increases the cost by about 25 %. Plated Broad Flange Beams are usually cheaper and always more convenient than double-webbed or "box" types of girders, especially where cross girders are used, or where girders have to be connected to stanchions. The larger plated sections 20 inches deep and upwards have a higher carrying power than B.F. Beam 30″ × 12″ and make excellent and economical substitutes for plate and angle girders.

AS STANCHIONS.

Plated Broad Flange Beams can be used with advantage in such cases as the following:—

(1) When the loads are comparatively heavy and the stanchion, if composed of a plain beam, would be inconveniently large.

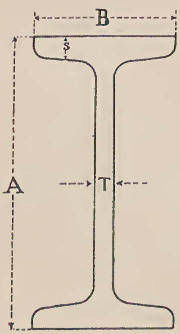
(2) When it is desired to employ the same size of stanchion throughout several floors, varying the number and thickness of the flange plates according to the loads. The requisite size of flange plates for this purpose can be ascertained with a fair degree of accuracy by assuming the carrying power to be increased by the flange plates in proportion to the increase in sectional area (or weight per foot).

If any of the sections when used as stanchions are heavily plated, it will be desirable to stiffen the web. The stiffeners to be shaped to the flanges and so arranged as to divide the length into three equal portions.

BRITISH STANDARD JOISTS.

LIST OF SIZES AND PROPERTIES.

Size. A	Size. B	Weight per foot.	Thickness. T	Thickness. S	Sectional Area.	Moments of Inertia. XX	Moments of Inertia. YY	Section Moduli. XX	Section Moduli. YY	Radii of Gyration. XX	Radii of Gyration. YY	Deflection Coefficient.	Reference No. and Code Word.	
Inches.		Lbs.	Ins.	Ins.	Sq. Ins.	Ins.	Ins.	Ins.	Ins.	Ins.	Ins.			
3	$\times 1\frac{1}{2}$	4	·16	·248	1·18	1·66	·124	1·11	·165	1·19	·325	·588	203	Bachelor
3	$\times 3$	$8\frac{1}{2}$	·20	·332	2·50	3·79	1·26	2·53	·841	1·23	·710	·588	206	Backbone
4	$\times 1\frac{3}{4}$	5	·17	·240	1·47	3·67	·194	1·84	·222	1·58	·363	·441	209	Bacon
4	$\times 3$	$9\frac{1}{2}$	·22	·336	2·80	7·53	1·28	3·76	·854	1·64	·677	·441	212	Badger
$4\frac{3}{4}$	$\times 1\frac{3}{4}$	$6\frac{1}{2}$	·18	·323	1·91	6·77	·263	2·85	·300	1·88	·371	·372	215	Bailie
5	$\times 3$	11	·22	·376	3·24	13·6	1·46	5·45	·974	2·05	·672	·352	218	Bakery
5	$\times 4\frac{1}{2}$	18	·29	·448	5·29	22·7	5·66	9·08	2·51	2·07	1·03	·352	221	Balance
6	$\times 3$	12	·26	·348	3·53	20·2	1·34	6·74	·892	2·40	·616	·294	224	Balcony
6	$\times 4\frac{1}{2}$	20	·37	·431	5·88	34·7	5·41	11·6	2·40	2·43	·959	·294	227	Balloon
6	$\times 5$	25	·41	·520	7·35	43·6	9·11	14·5	3·64	2·44	1·11	·294	230	Balsam
7	$\times 4$	16	·25	·387	4·71	39·2	3·41	11·2	1·71	2·89	·851	·252	233	Banana
8	$\times 4$	18	·28	·402	5·30	55·7	3·57	13·9	1·79	3·24	·821	·221	236	Bandage
8	$\times 5$	28	·35	·575	8·24	89·4	10·3	22·3	4·10	3·29	1·12	·221	239	Bandit
8	$\times 6$	35	·44	·597	10·3	111	17·9	27·6	5·98	3·28	1·32	·221	242	Banquet
9	$\times 4$	21	·30	·460	6·18	81·1	4·20	18·0	2·10	3·62	·824	·196	245	Baptism
9	$\times 7$	58	·55	·924	17·1	230	46·3	51·1	13·2	3·67	1·65	·196	248	Barber
10	$\times 5$	30	·36	·552	8·82	146	9·78	29·1	3·91	4·06	1·05	·177	251	Barley
10	$\times 6$	42	·40	·736	12·4	212	22·9	42·3	7·64	4·14	1·36	·177	254	Barnacle
10	$\times 8$	70	·60	·970	20·6	345	71·6	69·0	17·9	4·09	1·87	·177	257	Baronet
12	$\times 5$	32	·35	·550	9·41	220	9·74	36·7	3·90	4·84	1·02	·147	260	Barrack
12	$\times 6$	44	·40	·717	12·9	315	22·3	52·6	7·42	4·94	1·31	·147	263	Barrier
12	$\times 6$	54	·50	·883	15·9	376	28·3	62·6	9·43	4·86	1·33	·147	266	Barrister
14	$\times 6$	46	·40	·698	13·5	441	21·6	62·9	7·20	5·71	1·26	·126	269	Barrow
14	$\times 6$	57	·50	·873	16·8	533	27·9	76·2	9·31	5·64	1·29	·126	272	Basement
15	$\times 5$	42	·42	·647	12·4	428	11·9	57·1	4·78	5·89	·983	·118	275	Basilisk
15	$\times 6$	59	·50	·880	17·3	629	28·2	83·9	9·40	6·02	1·28	·118	278	Basket
16	$\times 6$	62	·55	·847	18·2	726	27·1	90·7	9·02	6·31	1·22	·110	281	Bassoon
18	$\times 7$	75	·55	·928	22·1	1150	46·6	128	13·3	7·22	1·45	·098	284	Bastion
20	$\times 7\frac{1}{2}$	89	·60	1·01	26·2	1671	62·6	167	16·7	7·99	1·55	·088	287	Bayonet
24	$\times 7\frac{1}{2}$	100	·60	1·07	29·4	2655	66·9	221	17·8	9·50	1·51	·074	290	Bazaar



RADII (C_1 and C_2).

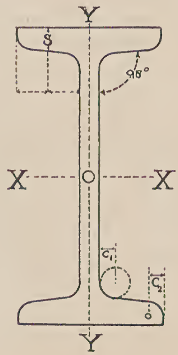
These can be readily ascertained as follows :—

C_1 = Web Thickness (T) + 0·1 inch.
$C_2 = \frac{1}{2} C_1$.

For example, the radii of a $5'' \times 4\frac{1}{2}''$ joist are ·390″ and ·195″ respectively.

TAPER.

The angle of 98° shown on key drawing corresponds to a taper of 14 % (say 1 in 7).



BRITISH STANDARD JOISTS.—Continued.

LIST OF SIZES AND SAFE DISTRIBUTED LOADS.

Size. A × B (Inches.)	Weight per foot. (Lbs.)	6′	8′	10′	12′	14′	16′	18′	20′	22′	24′	26′	28′	30′	32′	34′	36′	38′	40′
		SAFE DISTRIBUTED LOADS (TONS).																	
3 × 1½	4	0·9	0·7	0·6	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
3 × 3	8½	2·1	1·6	1·3	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
4 × 1¾	5	1·5	1·1	0·9	0·8	0·7	...	...	...	...	...	...	...	...	...	...	...	...	...
4 × 3	9½	3·1	2·4	1·9	1·6	1·3	...	...	...	...	...	...	...	...	...	...	...	...	...
4¾ × 1¾	6½	2·4	1·8	1·4	1·2	1·0	0·9	...	...	...	...	...	...	...	...	...	...	...	...
5 × 3	11	4·5	3·4	2·7	2·3	1·9	1·7	...	...	...	...	...	...	...	...	...	...	...	...
5 × 4½	18	7·6	5·7	4·5	3·8	3·2	2·8	...	...	...	...	...	...	...	...	...	...	...	...
6 × 3	12	5·6	4·2	3·4	2·8	2·4	2·1	1·9	...	...	...	...	...	...	...	...	...	...	...
6 × 4½	20	10	7·2	5·8	4·8	4·1	3·6	3·2	...	...	...	...	...	...	...	...	...	...	...
6 × 5	25	12	9·1	7·2	6·1	5·2	4·5	4·0	...	...	...	...	...	...	...	...	...	...	...
7 × 4	16	9·3	7·0	5·6	4·7	4·0	3·5	3·1	...	...	...	...	...	...	...	...	...	...	...
8 × 4	18	...	8·7	7·0	5·8	5·0	4·4	3·9	3·5	...	...	...	...	...	...	...	...	...	...
8 × 5	28	...	14	11	9·3	8·0	7·0	6·2	5·6	...	...	...	...	...	...	...	...	...	...
8 × 6	35	...	17	14	12	9·9	8·6	7·7	6·9	...	...	...	...	...	...	...	...	...	...
9 × 4	21	...	11	9·0	7·5	6·4	5·6	5·0	4·5	4·1	...	...	...	...	...	...	...	...	...
9 × 7	58	...	...	26	21	18	16	14	13	12	...	...	...	...	...	...	...	...	...
10 × 5	30	...	...	15	12	10	9·3	8·1	7·3	6·6	6·1	...	...	...	...	...	...	...	...
10 × 6	42	...	...	21	18	15	13	12	11	9·6	8·8	...	...	...	...	...	...	...	...
10 × 8	70	...	...	35	29	25	22	19	17	16	14	...	...	...	...	...	...	...	...
12 × 5	32	...	...	18	15	13	12	10	9·2	8·3	7·6	7·0	6·6	...	...	...	...	...	...
12 × 6	44	...	...	26	22	19	16	15	13	12	11	10	9·4	...	...	...	...	...	...
12 × 6	54	...	...	31	26	22	20	17	16	14	13	12	11	...	...	...	...	...	...
14 × 6	46	...	...	32	26	23	20	18	16	14	13	12	11	10	...	...	...	...	...
14 × 6	57	...	...	38	32	27	24	21	19	17	16	15	14	13	...	...	...	...	...
15 × 5	42	...	...	29	24	20	18	16	14	13	12	11	10	9·5	8·9	8·4	...	...	...
15 × 6	59	...	...	42	35	30	26	23	21	19	18	16	15	14	13	12	...	...	...
16 × 6	62	...	...	...	38	32	28	25	23	21	19	17	16	15	14	13	...	...	...
18 × 7	75	...	...	...	...	46	40	36	32	29	27	25	23	21	20	19	18	17	...
20 × 7½	89	...	...	...	...	60	52	46	42	38	35	32	30	28	26	25	23	22	21
24 × 7½	100	...	...	...	...	79	69	61	55	50	46	43	40	37	35	33	31	29	28

NOTES TO THE ABOVE TABLES.

PROPERTIES. For definitions, see page 8.

SAFE LOADS. These are calculated for a working stress of 7½ tons per square inch, by the formula :—Safe Load = 8 × Section Modulus (xx) × Working Stress ÷ Span (inches).

DEFLECTION. Loads to the right of the zig-zag line cause a deflection exceeding 1/360th of the span.

DEFLECTION COEFFICIENTS. These multiplied by the square of the span (feet) and divided by 100 give the deflection in inches corresponding to the tabulated safe loads. For further notes on deflection, see pages 137 to 139.

WIDE-FLANGED GIRDERS. The above sections can very often be replaced or supplemented with advantage by Broad Flange Beams, especially in such cases as the following :—(1) Crane-bearing Girders. (2) Where Girders are connected to Stanchions. See page 133. (3) Where it is desired to employ Girders over long spans so as to dispense with intermediate supports. (4) Where head-room is of importance, as in galleries in theatres etc. (5) Main Girders in Concrete Floors. For further information as to the uses and advantages of wide-flanged girders, see page 123.

GIRDERS WITHOUT SIDE SUPPORT. If the span exceeds 20 times the flange width, refer to page 141.

FLOOR GIRDERS. See page 142.

GIRDERS CARRYING BRICK WALLS. See page 128.

SHEARING STRESSES. See page 140.

CONCENTRATED LOADS Etc. See page 124.

BRITISH STANDARD JOISTS AS STRUTS OR STANCHIONS.

TABLE OF SAFE LOADS.

Size. A	Size. B	Weight per foot.	Sectional Area.	R_y	Safe loads in tons for heights or lengths of 4 to 20 feet. 4	6	8	10	12	16	20
Inches.		Lbs.	Sq. Ins.	Ins.							
3	× 1½	4	1·18	·325	2·4	...	...	...	...	...	...
3	× 3	8½	2·50	·710	11	8·4	6·0	4·3	...	...	...
4	× 1¾	5	1·47	·363	3·6	...	...	...	...	...	...
4	× 3	9½	2·80	·677	12	9·0	6·2	4·8	...	...	...
4¾	× 1¾	6½	1·91	·371	4·9	...	...	...	...	...	...
5	× 3	11	3·24	·672	14	10	7·1	5·1	...	...	...
5	× 4½	18	5·29	1·03	26	23	19	15	12	...	...
6	× 3	12	3·53	·616	15	10	6·8	...	...	...	...
6	× 4½	20	5·88	·959	28	25	20	16	12	...	...
6	× 5	25	7·35	1·11	37	33	29	23	19	12	...
7	× 4	16	4·71	·851	22	18	14	11	8·1	...	...
8	× 4	18	5·30	·821	25	20	15	11	8·6	...	...
8	× 5	28	8·24	1·12	41	37	32	26	21	14	...
8	× 6	35	10·3	1·32	52	48	44	39	32	22	...
9	× 4	21	6·18	·824	29	24	18	13	10	...	...
9	× 7	58	17·1	1·65	88	84	79	72	66	49	36
10	× 5	30	8·82	1·05	43	39	33	26	21	...	...
10	× 6	42	12·4	1·36	63	59	53	48	40	28	20
10	× 8	70	20·6	1·87	107	104	98	93	85	68	52
12	× 5	32	9·41	1·02	46	40	34	27	21	...	...
12	× 6	44	12·9	1·31	66	61	55	48	40	27	...
12	× 6	54	15·9	1·33	81	75	68	60	50	35	25
14	× 6	46	13·5	1·26	69	63	56	49	40	27	...
14	× 6	57	16·8	1·29	85	79	71	62	50	35	...
15	× 5	42	12·4	·983	60	52	42	34	26	...	...
15	× 6	59	17·3	1·28	88	81	73	63	52	35	...
16	× 6	62	18·2	1·22	92	85	75	63	52	34	...
18	× 7	75	22·1	1·45	113	106	98	88	76	55	39
20	× 7½	89	26·2	1·55	135	127	119	108	97	70	51
24	× 7½	100	29·4	1·51	151	142	132	120	106	76	55

NOTES TO THE ABOVE TABLE.

(1) The above table was calculated by the same formula as Table A of safe loads on Broad Flange Beams, page 157. ("Fidler's." Factor of Safety, 5.) On comparing the above loads with those given in Table A, it will be observed that all narrow-flanged joist sections that weigh over 30 lbs. per foot can be replaced by Broad Flange Beams of greater carrying power and less weight. See also notes on the "Comparative Values of Different Types of Stanchions," pages 163 etc.

(2) R_y denotes the least radius of gyration (inches).

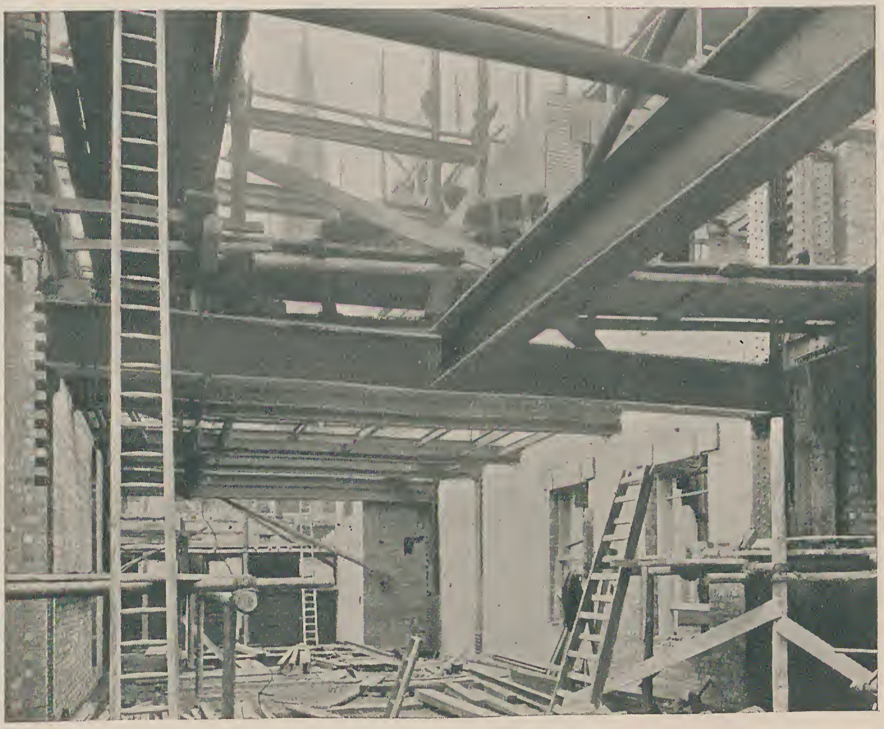
(3) If stanchions are selected from the above table to carry loads which are wholly or partly eccentric, the tabular loads may have to be reduced by 60 % or more. Such cases can be dealt with by the formulæ given on page 174 for eccentrically loaded stanchions.

BROAD FLANGE BEAMS IN BUILDING CONSTRUCTION.

BROAD FLANGE BEAMS 22″×12″ AS FLOOR GIRDERS IN MESSRS. J. LYONS & CO.'S POPULAR CAFÉ, PICCADILLY.

Architect:
W. J. ANCELL, Esq., London.

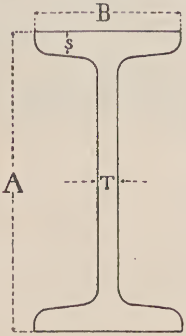
Consulting Engineers:
Messrs. READE, JACKSON & PARRY.



METRIC STANDARD JOISTS.

LIST OF SIZES AND PROPERTIES.

ENGLISH DIMENSIONS.									METRIC DIMENSIONS.			
Size.	Weight per foot.	Thickness.		Sectional Area.	Moments of Inertia.		Section Modulus (XX).	Least Radius of Gyration (YY).	Size.	Weight per metre.	Thickness.	
A × B		Web.	Flange.		XX	YY			A × B		Web.	Flange.
Inches.	Lbs.	Ins.	Ins.	Ins²	Ins.	Ins.	Ins.	Ins.	Mm.	Kilos.	Mm.	Mm.
3·15 × 1·65	3·97	·154	·232	1·17	1·87	·151	1·18	·359	80 × 42	5·91	3·9	5·9
3·54 × 1·81	4·72	·165	·248	1·39	2·81	·211	1·58	·389	90 × 46	7·02	4·2	6·3
3·94 × 1·97	5·56	·177	·268	1·64	4·08	·293	2·08	·423	100 × 50	8·28	4·5	6·8
4·33 × 2·13	6·44	·189	·283	1·91	5·72	·389	2·64	·452	110 × 54	9·59	4·8	7·2
4·72 × 2·28	7·46	·201	·303	2·20	7·86	·514	3·33	·483	120 × 58	11·1	5·1	7·7
5·12 × 2·44	8·47	·213	·319	2·50	10·5	·659	4·09	·514	130 × 62	12·6	5·4	8·1
5·51 × 2·60	9·54	·224	·339	2·82	13·7	·846	4·99	·548	140 × 66	14·2	5·7	8·6
5·91 × 2·76	10·7	·236	·354	3·16	17·6	1·05	5·97	·576	150 × 70	15·9	6·0	9·0
6·30 × 2·91	12·0	·248	·374	3·53	22·4	1·31	7·14	·609	160 × 74	17·8	6·3	9·5
6·69 × 3·07	13·2	·260	·390	3·91	28·0	1·60	8·36	·640	170 × 78	19·7	6·6	9·9
7·09 × 3·23	14·6	·272	·409	4·32	34·7	1·95	9·82	·672	180 × 82	21·7	6·9	10·4
7·48 × 3·39	16·0	·283	·425	4·73	42·3	2·34	11·3	·703	190 × 86	23·8	7·2	10·8
7·87 × 3·54	17·5	·295	·445	5·18	51·4	2·81	13·1	·737	200 × 90	26·1	7·5	11·3
8·27 × 3·70	19·0	·307	·461	5·63	61·5	3·29	14·9	·765	210 × 94	28·3	7·8	11·7
8·66 × 3·86	20·7	·319	·480	6·12	73·4	3·92	17·0	·800	220 × 98	30·8	8·1	12·2
9·06 × 4·02	22·4	·331	·496	6·60	86·6	4·52	19·2	·827	230 × 102	33·3	8·4	12·6
9·45 × 4·17	24·1	·343	·516	7·15	102	5·29	21·5	·860	240 × 106	35·9	8·7	13·1
9·84 × 4·33	26·0	·354	·535	7·70	119	6·13	24·2	·892	250 × 110	38·7	9·0	13·6
10·24 × 4·45	28·0	·370	·555	8·26	138	6·90	26·9	·914	260 × 113	41·6	9·4	14·1
10·63 × 4·57	29·9	·382	·579	8·85	159	7·81	30·0	·939	270 × 116	44·5	9·7	14·7
11·02 × 4·69	32·0	·398	·598	9·45	182	8·72	33·0	·961	280 × 119	47·6	10·1	15·2
11·42 × 4·80	34·0	·409	·618	10·0	207	9·68	36·2	·982	290 × 122	50·6	10·4	15·7
11·81 × 4·92	36·1	·425	·638	10·7	235	10·8	39·8	1·00	300 × 125	53·8	10·8	16·2
12·60 × 5·16	40·7	·453	·681	12·0	300	13·3	47·7	1·05	320 × 131	60·6	11·5	17·3
13·39 × 5·39	45·4	·480	·720	13·4	377	16·1	56·3	1·10	340 × 137	67·6	12·2	18·3
14·17 × 5·63	50·9	·512	·768	15·0	470	19·6	66·4	1·14	360 × 143	75·7	13·0	19·5
14·96 × 5·87	56·0	·539	·807	16·6	576	23·4	77·0	1·19	380 × 149	83·4	13·7	20·5
15·75 × 6·10	61·7	·567	·850	18·3	701	27·9	89·0	1·23	400 × 155	91·8	14·4	21·6
16·73 × 6·42	69·2	·602	·906	20·5	888	34·4	106	1·30	425 × 163	103	15·3	23·0
17·72 × 6·69	77·3	·638	·957	22·8	1103	41·4	125	1·35	450 × 170	115	16·2	24·3
18·70 × 7·01	85·3	·673	1·01	25·3	1355	50·1	145	1·41	475 × 178	127	17·1	25·6
19·69 × 7·28	94·1	·709	1·06	27·7	1652	59·3	168	1·46	500 × 185	140	18·0	27·0
21·65 × 7·87	112	·748	1·18	32·9	2380	83·8	220	1·60	550 × 200	166	19·0	30·0
23·62 × 8·46	134	·850	1·28	39·4	3339	...	283	...	600 × 215	199	21·6	32·4



RADII (C_1 and C_2).

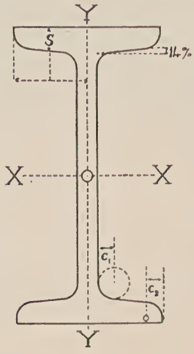
These can be readily ascertained as follows :—

C_1 = Web Thickness.
C_2 = 0·6 × Web Thickness.

Except in the case of Section No. 396, for which the radii are 19·8 mm. and 11·9 mm. respectively.

TAPER.

The taper of 14 % shown on key drawing corresponds to an angle of 98° between flange and web.



METRIC STANDARD JOISTS.—Continued.

LIST OF SIZES AND SAFE LOADS.

Size in inches. A × B	SAFE UNIFORMLY DISTRIBUTED LOADS (IN ENGLISH TONS) FOR VARIOUS SPANS (IN FEET). 10′	12′	14′	16′	18′	20′	22′	24′	28′	32′	36′	Reference No.	and Code Word.
3·15 × 1·65	·59	·49	·42	·37	·33	·30	...	...	...	...	...	310	Cactus
3·54 × 1·81	·79	·66	·56	·49	·44	·40	...	...	...	...	...	313	Caitiff
3·94 × 1·97	1·0	·87	·74	·65	·58	·52	...	...	...	...	...	316	Caliph
4·33 × 2·13	1·3	1·1	·94	·83	·73	·66	...	...	...	...	...	319	Cambric
4·72 × 2·28	1·7	1·4	1·2	1·0	·93	·83	...	...	...	...	...	322	Camera
5·12 × 2·44	2·0	1·7	1·5	1·3	1·1	1·0	...	...	...	...	...	325	Camphor
5·51 × 2·60	2·5	2·1	1·8	1·6	1·4	1·2	...	...	...	...	...	328	Canary
5·91 × 2·76	3·0	2·5	2·1	1·9	1·7	1·5	1·4	...	...	...	...	331	Candy
6·30 × 2·91	3·6	3·0	2·6	2·2	2·0	1·8	1·6	1·5	...	...	...	334	Cannibal
6·69 × 3·07	4·2	3·5	3·0	2·6	2·3	2·1	1·9	1·7	...	...	...	337	Cannon
7·09 × 3·23	4·9	4·1	3·5	3·1	2·7	2·5	2·2	2·0	...	...	...	340	Canopy
7·48 × 3·39	5·7	4·7	4·0	3·5	3·1	2·8	2·6	2·4	2·0	...	...	343	Canticle
7·87 × 3·54	6·6	5·5	4·7	4·1	3·6	3·3	3·0	2·7	2·3	...	...	346	Canvas
8·27 × 3·70	7·5	6·2	5·3	4·7	4·1	3·7	3·4	3·1	2·7	...	...	349	Caper
8·66 × 3·86	8·5	7·1	6·1	5·3	4·7	4·3	3·9	3·5	3·0	2·7	...	352	Caprice
9·06 × 4·02	9·6	8·0	6·9	6·0	5·3	4·8	4·4	4·0	3·4	3·0	...	355	Capstan
9·45 × 4·17	11	9·0	7·7	6·7	6·0	5·4	4·9	4·5	3·8	3·4	3·0	358	Capsule
9·84 × 4·33	12	10	8·6	7·6	6·7	6·1	5·5	5·0	4·3	3·8	3·4	361	Captor
10·24 × 4·45	13	11	9·6	8·4	7·5	6·7	6·1	5·6	4·8	4·2	3·7	364	Caramel
10·63 × 4·57	15	13	11	9·4	8·3	7·5	6·8	6·3	5·4	4·7	4·2	367	Caravan
11·02 × 4·69	17	14	12	10	9·2	8·3	7·5	6·9	5·9	5·2	4·6	370	Carbine
11·42 × 4·80	18	15	13	11	10	9·1	8·2	7·5	6·5	5·7	5·0	372	Carcass
11·81 × 4·92	20	17	14	12	11	10	9·0	8·3	7·1	6·2	5·5	374	Cardinal
12·60 × 5·16	24	20	17	15	13	12	11	9·9	8·5	7·5	6·6	376	Carrot
13·39 × 5·39	28	24	20	18	16	14	13	12	10	8·8	7·8	378	Carver
14·17 × 5·63	33	28	24	21	18	17	15	14	12	10	9·2	380	Casino
14·96 × 5·87	39	32	28	24	21	19	18	16	14	12	11	382	Casket
15·75 × 6·10	45	37	32	28	25	22	20	19	16	14	12	384	Cassock
16·73 × 6·42	...	44	38	33	29	27	24	22	19	17	15	386	Caterer
17·72 × 6·69	...	52	45	39	35	31	28	26	22	20	17	388	Catholic
18·70 × 7·01	...	61	52	45	40	36	33	30	26	23	20	390	Cavalier
19·69 × 7·28	...	...	60	53	47	42	38	35	30	26	23	392	Cayenne
21·65 × 7·87	...	...	79	69	61	55	50	46	39	34	31	396	Cereal
23·62 × 8·46	...	...	...	88	79	70	64	59	50	44	39	398	Ceremony

NOTES TO THE ABOVE TABLES.

(1) The safe loads are calculated for a working stress (or "extreme fibre stress") of $7\frac{1}{2}$ tons per square inch (11·81 kilos per square mm.) by the formula:—

Safe Load = 8 × Section Modulus (xx) × Working Stress ÷ Span (inches).

(2) Loads to the right of the zig-zag line cause a deflection exceeding 1/360th of the span.

(3) The largest size in the tables (Section No. 398) is not strictly standard nor at present freely obtainable.

(4) The weights as given above are those given by most Continental manufacturers and are based on an assumed weight of 3·381 lbs. per foot per square inch of cross section. Actually, this is the average weight of wrought-iron, that of steel being 0·64 % greater, viz. 3·4 lbs. per foot per square inch. In view of the usual rolling margin claimed, the difference is almost immaterial.

Misc. Shapes.
Misc. Tables.
Prices etc.
Code.
Grey Mill.
Tests etc.
R.I.B.A.
Photos.

BRITISH STANDARD CHANNELS.

LIST OF SIZES AND PROPERTIES.

Size.		Weight per foot.		Thickness.		Radii.		Sectional Area.	G	Moments of Inertia.		Section Moduli.		Reference No. and Code Word.	
A	B			T	S	C_1	C_2			XX	YY	XX	YY		
Inches.		Lbs.		Ins.	Ins.	Ins.	Ins.	Sq. Ins.	Ins.	Ins.	Ins.	Ins.	Ins.		
3	× 1½	5·27	*c*	·250	·312	·312	·220	1·55	0·48	1·99	·296	1·33	·291	406	Daffodil
3½	× 2	6·75	*c*	·250	·312	·312	·220	1·99	0·65	3·70	·713	2·12	·526	409	Dagger
4	× 2	7·96	*a*	·250	·375	·375	·260	2·34	0·66	5·71	·843	2·86	·627	412	Dakoit
5	× 2½	11·0	*b*	·312	·375	·375	·260	3·23	0·76	12·1	1·77	4·85	1·02	415	Damask
6	× 2½	12·0	*b*	·312	·375	·375	·260	3·54	0·70	18·8	1·88	6·25	1·05	418	Dandy
6	× 3	14·5	*c*	·312	·437	·437	·300	4·26	0·94	24·0	3·50	8·00	1·70	421	Deacon
6	× 3	16·3	*a*	·375	·475	·475	·325	4·79	0·93	26·0	3·82	8·68	1·85	424	Decoction
6	× 3½	17·9	*c*	·375	·475	·475	·325	5·27	1·12	29·7	5·91	9·89	2·48	427	Decorum
7	× 3	17·6	*	·375	·475	·475	·325	5·17	0·87	37·6	4·02	10·8	1·89	430	Decoy
7	× 3½	20·2	*b*	·400	·500	·500	·350	5·95	1·06	44·5	6·50	12·7	2·66	433	Defence
8	× 2½	15·1	*	·312	·437	·437	·300	4·45	0·67	41·1	2·28	10·3	1·25	436	Democrat
8	× 3	19·3	*c*	·375	·500	·500	·350	5·68	0·84	53·4	4·33	13·4	2·01	439	Demon
8	× 3½	22·7	*a*	·425	·525	·525	·375	6·68	1·01	63·8	7·07	15·9	2·84	442	Dentist
8	× 4	25·7	*d*	·450	·550	·550	·375	7·57	1·20	74·0	10·8	18·5	3·86	445	Derision
9	× 3	19·4	*a*	·375	·437	·437	·350	5·70	0·75	65·2	4·02	14·5	1·79	448	Despot
9	× 3½	22·3	*b*	·375	·500	·500	·350	6·55	0·98	79·9	6·96	17·8	2·76	451	Directory
9	× 3½	25·4	*c*	·450	·550	·550	·375	7·47	0·97	88·1	7·66	19·6	3·03	454	Discourse
9	× 4	28·6	*d*	·475	·575	·575	·400	8·40	1·15	102	11·6	22·6	4·08	457	Disgrace
10	× 3½	23·6	*	·375	·500	·500	·350	6·93	0·93	103	7·19	20·5	2·80	460	Disguise
10	× 3½	28·2	*	·475	·575	·575	·400	8·30	0·93	118	8·19	23·6	3·19	463	Dispenser
10	× 4	30·2	*b*	·475	·575	·575	·400	8·87	1·10	131	12·0	26·1	4·15	466	Display
11	× 3½	29·8	*c*	·475	·575	·575	·400	8·77	0·90	149	8·42	27·0	3·23	469	Ditch
11	× 4	33·2	*d*	·500	·600	·600	·425	9·77	1·06	170	12·8	31·0	4·36	472	Divinity
12	× 3½	26·1	*d*	·375	·500	·500	·350	7·68	0·86	159	7·57	26·4	2·87	475	Doctor
12	× 3½	32·9	*a*	·500	·600	·600	·425	9·67	0·87	191	8·92	31·8	3·39	478	Doeskin
12	× 4	36·5	*b*	·525	·625	·625	·425	10·7	1·03	218	13·7	36·4	4·60	481	Dogma
15	× 4	41·9	*a*	·525	·630	·630	·440	12·3	0·94	377	14·6	50·3	4·75	484	Domicile

LIST OF SIZES.

(1) Sizes marked "*a*" are promptly obtainable in any quantities.

Sizes marked "*b*" are in common demand and frequently rolled.

Sizes marked "*c*" are rolled less frequently by a limited number of manufacturers.

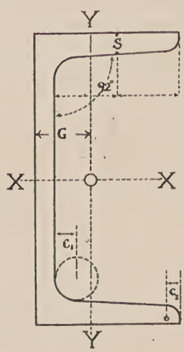
Sizes marked "*d*" are not readily obtainable at present and should be avoided.

Of sizes marked with an asterisk (*), 7″ × 3″ channels are at present readily obtainable ½″ thick, but not ⅜″ thick. 8″ × 2½″ channels, when rolls are new, really measure 200 mm. deep (7⅞″). 10″ × 3½″ channels are readily obtainable from stock, but the sections stocked by different firms vary in weight from 23 to 28 lbs. per foot and, in thickness, from ⅜″ to ½″.

(2) Besides the above standard sizes of channels, many other sizes are made for special purposes, including very small sizes such as ⅜″ × ⅜″ × ⅛″.

TAPER.

The angle of 92° shown on drawing corresponds to a taper of 3½ % (say 1 in 28).



BRITISH STANDARD CHANNELS.—Continued.

SAFE LOADS AS STRUTS.

Size.		Weight per foot.	Radii of Gyration.		If liable to failure in either direction.						If braced against failure in weaker direction.					
A	B		XX	YY	6′	8′	10′	12′	14′	16′	6′	8′	10′	12′	14′	16′
Inches.		Lbs.	Ins.	Ins.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.
3	× 1½	5·27	1·14	·437	2·73	...	...	...	...	...	7·02	6·14	5·01	4·08	3·27	2·65
3½	× 2	6·75	1·37	·599	5·55	3·68	...	...	...	...	9·43	8·62	7·66	6·43	5·39	4·52
4	× 2	7·96	1·56	·600	6·53	4·33	...	...	...	...	11·4	10·7	9·69	8·73	7·35	6·34
5	× 2½	11·0	1·94	·741	11·3	8·14	5·88	...	...	...	16·4	15·6	14·7	13·6	12·5	11·1
6	× 2½	12·0	2·30	·729	12·1	8·74	6·23	...	...	...	18·2	17·7	16·9	16·0	15·0	14·1
6	× 3	14·5	2·37	·907	17·5	13·6	10·5	7·97	...	...	22·0	21·4	20·4	19·6	18·3	17·3
6	× 3	16·3	2·33	·893	19·4	15·0	11·5	8·77	...	...	24·7	24·1	22·8	21·8	20·5	19·3
6	× 3½	17·9	2·37	1·06	23·1	19·8	15·7	12·4	9·85	...	27·3	26·5	25·2	24·2	22·7	21·3
7	× 3	17·6	2·70	·882	20·8	16·1	12·2	9·31	...	...	26·9	26·4	25·5	24·5	23·6	22·2
7	× 3½	20·2	2·74	1·05	25·9	22·4	17·5	13·9	11·0	...	31·0	30·4	29·4	28·2	27·3	25·8
8	× 2½	15·1	3·04	·716	14·9	10·7	7·65	...	...	...	23·3	22·9	22·5	21·7	20·9	20·2
8	× 3	19·3	3·07	·873	22·8	17·5	13·1	10·0	...	...	29·8	29·3	28·7	27·7	26·7	25·7
8	× 3½	22·7	3·09	1·03	28·9	24·7	19·1	15·2	12·0	...	35·0	34·4	33·8	32·5	31·5	30·5
8	× 4	25·7	3·13	1·19	34·7	30·7	25·4	21·0	17·0	13·9	39·7	39·0	38·5	37·0	35·7	34·7
9	× 3	19·4	3·38	·840	22·2	16·8	12·5	9·58	...	...	30·0	29·6	29·1	28·3	27·4	26·6
9	× 3½	22·3	3·49	1·03	28·4	24·2	18·7	14·9	11·8	...	34·5	34·0	33·5	32·9	31·7	30·8
9	× 3½	25·4	3·43	1·01	32·1	27·0	21·0	16·4	13·0	...	39·3	38·8	38·2	37·3	36·0	35·0
9	× 4	28·6	3·48	1·18	38·6	34·0	27·9	23·0	18·7	15·1	44·2	43·6	43·0	42·2	40·7	39·5
10	× 3½	23·6	3·85	1·02	29·8	25·3	19·6	15·6	12·2	...	36·5	36·2	35·7	35·3	34·2	33·3
10	× 3½	28·2	3·77	·994	35·4	29·1	23·0	17·9	14·2	...	43·7	43·3	42·7	42·2	40·8	39·7
10	× 4	30·2	3·84	1·16	40·4	35·4	28·9	23·8	19·2	15·4	46·7	46·3	45·7	45·1	43·8	42·6
11	× 3½	29·8	4·12	·980	37·3	30·4	23·8	18·5	14·7	...	46·2	46·0	45·4	44·8	44·0	42·7
11	× 4	33·2	4·18	1·15	44·3	39·0	31·9	25·9	20·8	16·9	51·5	51·2	50·6	50·0	49·3	47·8
12	× 3½	26·1	4·55	·993	32·6	27·0	21·3	16·6	13·1	...	40·5	40·4	40·0	39·5	39·1	38·3
12	× 3½	32·9	4·44	·960	40·5	32·8	25·6	19·6	15·8	...	51·0	50·8	50·3	49·7	49·1	48·1
12	× 4	36·5	4·51	1·13	48·2	42·1	34·2	27·6	22·1	18·2	56·4	56·3	55·6	55·0	54·5	53·2
15	× 4	41·9	5·53	1·09	54·6	47·4	37·9	30·4	24·2	19·8	64·8	64·8	64·6	64·1	63·6	62·9

SAFE DISTRIBUTED LOADS ON CHANNELS.

(1) To ascertain what size of channel will carry a given uniformly distributed load, when used as a beam :—Multiply the load to be carried (tons) by the span (feet). Divide the result by 5 for a working stress of 7½ tons per square inch or by 4 for a working stress of 6 tons per square inch. The result gives the required "Section Modulus" (axis XX if web is vertical, axis YY if web is horizontal).

(2) The above rule is for transverse bending. Liability to shear and lateral failure must also be considered.

TABULATED SAFE LOADS ON CHANNELS AS STRUTS.

The safe loads given above apply to single channels used as struts with axis vertical and load centrally applied. They are calculated by the same formula as Table A of safe loads on Broad Flange Beams (Fidler's Formula, Fixed Ends, Factor of Safety 5).

The loads on the right-hand side of the page are calculated with respect to the *greatest* radius of gyration (axis XX).

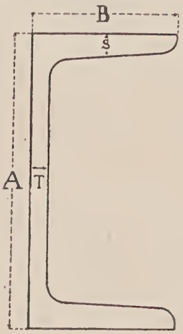
Misc. Shapes.
Misc. Tables.
Prices etc.
Code.
Grey Mill.
Tests etc.
R.I.B.A.
Photos.

METRIC STANDARD CHANNELS.

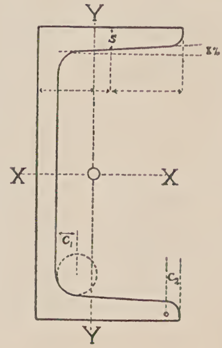
LIST OF SIZES AND PROPERTIES.

ENGLISH DIMENSIONS.								Reference No. and Code Word.		METRIC DIMENSIONS.			
Size. A × B	Weight per foot.	Thickness. T	Thickness. S	Sectional Area.	Moments of Inertia. XX	Moments of Inertia. YY	Section Modulus. (XX).			Size. A × B	Weight per metre.	Thickness. T	Thickness. S
Inches.	Lbs.	Ins.	Ins.	Sq. Ins.	Ins.	Ins.	Ins.			Mm.	Kilos.	Mm.	Mm.
1·18 × 1·30	2·85	·197	·276	·843	·154	·128	·260	510	Eagle	30 × 33	4·24	5	7
1·57 × 1·38	3·26	·197	·276	·963	·339	·161	·433	514	Earth	40 × 35	4·85	5	7
1·97 × 1·50	3·73	·197	·276	1·10	·634	·219	·647	518	Easel	50 × 38	5·55	5	7
2·56 × 1·65	4·74	·217	·295	1·40	1·38	·339	1·08	522	Easement	65 × 42	7·05	5·5	7·5
3·15 × 1·77	5·78	·236	·315	1·71	2·55	·466	1·62	526	Ebony	80 × 45	8·6	6	8
3·94 × 1·97	7·06	·236	·335	2·09	4·95	·704	2·51	530	Eclipse*	100 × 50	10·5	6	8·5
4·13 × 2·56	9·07	·315	·315	2·68	6·90	1·47	3·34	534	Edifice	105 × 65	13·5	8	8
4·63 × 2·56	11·8	·394	·394	3·50	10·7	1·85	4·64	538	Edition	117½ × 65	17·6	10	10
4·72 × 2·17	8·94	·276	·354	2·63	8·75	1·04	3·70	542	Editor*	120 × 55	13·3	7	9
5·51 × 2·36	10·7	·276	·394	3·16	14·5	1·51	5·27	546	Effigy*	140 × 60	15·9	7	10
5·71 × 2·36	10·4	·315	·315	3·07	14·1	1·29	4·93	550	Effort	145 × 60	15·4	8	8
6·30 × 2·56	12·6	·295	·413	3·72	22·2	2·05	7·08	554	Egotist*	160 × 65	18·7	7·5	10·5
7·09 × 2·76	14·7	·315	·433	4·34	32·5	2·74	9·15	558	Egrette*	180 × 70	21·8	8	11
7·87 × 2·95	16·9	·335	·453	4·99	45·9	3·56	11·7	562	Ejection*	200 × 75	25·1	8·5	11·5
8·66 × 3·15	19·6	·354	·492	5·80	64·7	4·73	15·0	566	Elation*	220 × 80	29·2	9	12·5
9·25 × 3·54	22·2	·394	·472	6·57	82·4	6·54	17·8	570	Elegy	235 × 90	33·1	10	12
9·45 × 3·35	22·2	·374	·512	6·56	86·5	5·96	18·3	574	Elixir*	240 × 85	33	9·5	13
10·24 × 3·54	21·8	·394	·394	6·45	93·7	5·69	18·3	578	Ellipse*	260 × 90	32·5	10	10
10·24 × 3·54	25·3	·394	·551	7·49	116	7·62	22·6	582	Elysium	260 × 90	37·7	10	14
11·02 × 3·74	28·0	·394	·591	8·26	151	9·59	27·5	586	Embassy	280 × 95	41·6	10	15
11·81 × 2·95	22·4	·394	·394	6·63	118	3·48	20·0	590	Emblem	300 × 75	33·3	10	10
11·81 × 3·94	30·8	·394	·630	9·11	193	11·9	32·7	594	Embryo*	300 × 100	45·8	10	16

All of the above sizes are rolled by various Continental manufacturers. Those marked with an asterisk (*) are the commonest sizes used. In addition to the standard sizes given in the table, a number of additional sizes are made at various works for use in the underframes of railway waggons, but being in less frequent demand, prompt delivery of small lots cannot be depended upon.



The taper of 8 % corresponds to an angle of 94½° between web and flange.



STEEL PIT-HEAD FRAME.

The above photograph shows a light steel pit frame designed and constructed by Messrs. Rees & Kirby, Ltd., Steel Roof Builders, Morriston, R.S.O., Glam.

The front and main legs are Broad Flange Beams of section $9\frac{1}{2}'' \times 9\frac{1}{2}''$; the back stays are of section $11'' \times 11''$.

Messrs. Rees & Kirby have made heavier frames with Broad Flange Beams in like fashion.

BRITISH STANDARD EQUAL-SIDED ANGLES.

LIST OF SIZES AND PROPERTIES.

[For Key Drawing, P.T.O.]

Size. A × B	Thickness.		Weight per foot.	Reference No. and Code Word.		Sectional Area.	Radii. C_1	Radii. C_2	Moments of Inertia. XX	Moments of Inertia. UU	Moments of Inertia. VV	Least Radius of Gyration.	Section Modulus. (xx).
Inches.	Ins.		Lbs.			Sq. Ins.	Ins.	Ins.		Max.	Min.	Ins.	Ins.
1 × 1	⅛		·80	602	Fabian	·234	·175	·125	·020	·032	·008	·185	·028
,,	¼		1·49	604	Fabric	·437	..	..	..	..	..	..	..
1¼ × 1¼	⅛		1·02	606	Factor	·299	·200	·150	·041	·065	·017	·238	·045
,,	¼		1·92	608	Fairy	·564	..	..	..	..	..	..	..
1½ × 1½	⅛		1·23	610	Falcon	·361	·200	·150	·074	·117	·031	·293	·068
,,	¼	*a*	2·34	612	Family	·689	..	..	..	..	..	..	..
1¾ × 1¾	·175		1·98	614	Fancier	·583	·225	·150	·162	·257	·067	·339	·129
,,	¼*		2·76	616	Fantasy	..	..	..	..	..	..	..	..
,,	·300		3·27	618	Fatality	·961	..	..	..	..	..	..	..
2 × 2	·175		2·28	620	Fatigue	·670	·250	·175	·244	·387	·101	·388	·168
,,	¼*	*a*	3·19	622	Favourite	..	..	..	..	..	..	..	..
,,	·300		3·77	624	Feast	1·11	..	..	..	..	..	..	..
2¼ × 2¼	·175		2·57	626	Felicity	·757	·250	·175	·354	·562	·146	·439	·216
,,	¼*		3·61	628	Fence	..	..	..	..	..	..	..	..
,,	·300		4·28	630	Ferret	1·26	..	..	..	..	..	..	..
2½ × 2½	¼	*a*	4·04	632	Fetter	1·19	·275	·200	·677	1·08	·278	·485	·377
,,	⅜	*a*	5·89	634	Fever	1·73	..	..	..	..	..	..	..
,,	½		7·65	636	Fiction	2·25	..	..	..	..	..	..	..
2¾ × 2¾	¼		4·46	638	Fiddler	1·31	·275	·200	·917	1·46	·377	·535	·462
,,	⅜		6·53	640	Figment	1·92	..	..	..	..	..	..	..
,,	½		8·50	642	Filigree	2·50	..	..	..	..	..	..	..
3 × 3	¼		4·90	644	Filter	1·44	·300	·200	1·21	1·92	·495	·587	·555
,,	⅜	*a*	7·18	648	Finale	2·11	..	..	..	..	..	..	..
,,	½	*a*	9·36	650	Finery	2·75	..	..	..	..	..	..	..
3½ × 3½	·300		6·84	652	Finesse	2·01	·325	·225	2·30	3·66	·942	·684	·908
,,	⅜*	*a*	8·45	654	Fishery	..	..	..	..	..	..	..	..
,,	·425		9·50	656	Flambeau	2·80	..	..	..	..	..	..	..
,,	½	*a*	11·1	658	Flavour	3·25	..	..	..	..	..	..	..
4 × 4	·300		7·85	660	Fleece	2·31	·350	·250	3·48	5·53	1·42	·785	1·20
,,	⅜*	*a*	9·72	662	Flesh	..	..	..	..	..	..	..	..
,,	·425		10·9	664	Flint	3·22	..	..	..	..	..	..	..
,,	½	*a*	12·8	666	Float	3·75	..	..	..	..	..	..	..
,,	⅝*	*a*	15·7	667	Flock	..	..	..	..	..	..	..	..
4½ × 4½	⅜		11·0	668	Flora	3·22	·400	·275	6·14	9·77	2·51	·882	1·89
,,	½	*a*	14·5	670	Florist	4·25	..	..	..	..	..	..	..
5 × 5	⅜		12·3	672	Flotilla	3·61	·425	·300	8·51	13·5	3·48	·982	2·34
,,	½	*a*	16·2	674	Flower	4·75	..	..	..	..	..	..	..
6 × 6	·450		17·7	676	Flunky	5·20	·475	·325	17·7	28·2	7·25	1·18	4·07
,,	½*	*a*	19·6	678	Flute	..	..	..	..	..	..	..	..
,,	⅝	*a*	24·2	680	Fodder	7·11	..	..	..	..	..	..	..
7 × 7	½		23·0	682	Forceps	6·76	·550	·375	31·4	50·1	12·8	1·38	6·18
,,	⅝*		28·4	684	Foresail	..	..	..	..	..	..	..	..
,,	·675		30·6	686	Forester	9·00	..	..	..	..	..	..	..
8 × 8	·550		28·9	688	Fortress	8·50	·600	·425	51·8	82·5	21·1	1·58	8·89
,,	⅝*		32·7	690	Fountain	..	..	..	..	..	..	..	..
,,	¾		38·9	692	Frivolity	11·4	..	..	..	..	..	..	..

NOTES TO THE ABOVE TABLES.

(1) Sizes marked "*a*" are freely stocked in various thicknesses. 7″×7″ angles are not readily obtainable and should be avoided. 8″×8″ angles are not in great demand.

(2) The odd thicknesses given in decimals of an inch, although standard, are intended mainly for shipbuilding and are preferably avoided by the ordinary user.

(3) Thicknesses marked with an asterisk (*) are not standard thicknesses but are readily obtainable. All other intermediate thicknesses can be supplied, and most of the sizes are also rolled in thicknesses exceeding the maximum thickness given in the table.

(4) Code Words.—In the case of thicknesses not given in the table, the above code words can

[Continued on page opposite.]

BRITISH STANDARD EQUAL-SIDED ANGLES.—Continued.

LIST OF SIZES AND SAFE LOADS.

[For Key Drawing, P.T.O.]

Size. A × B	Thickness.	SAFE DISTRIBUTED LOADS (TONS) FOR VARIOUS SPANS (FEET). 4′	5′	6′	7′	8′	9′	10′	11′	12′	13′	SAFE LOADS (TONS) AS STRUTS. HEIGHTS GIVEN IN FEET. 4′	6′	8′	10′	12′	14′	16′	18′	20′
Inches.	Ins.																			
1 × 1	$\frac{1}{8}$	·04	·03	·02	..	..	..	..	..	..	..	·20	·09	..	..	..	..	..	..	..
,,	$\frac{1}{4}$	·07	·05	·04	·04	..	..	..	..	..	..	·37	·16	..	..	..	..	..	..	..
$1\frac{1}{4} \times 1\frac{1}{4}$	$\frac{1}{8}$	·06	·05	·04	·03	·03	..	..	..	..	..	·40	·18	..	..	..	..	..	..	..
,,	$\frac{1}{4}$	·11	·09	·07	·06	·05	·05	..	..	..	..	·75	·34	..	..	..	..	..	..	..
$1\frac{1}{2} \times 1\frac{1}{2}$	$\frac{1}{8}$	·09	·07	·06	·05	·04	·04	·03	..	..	..	·64	·34	·18	..	..	..	..	..	..
,,	$\frac{1}{4}$	·16	·13	·11	·09	·08	·07	·07	·06	·05	·05	1·2	·66	·35	..	..	..	..	..	..
$1\frac{3}{4} \times 1\frac{3}{4}$	·175	·16	·13	·11	·09	·08	·07	·06	·06	·05	·05	1·3	·71	·41	·24	..	..	..	..	..
,,	$\frac{1}{4}$	·22	·18	·15	·13	·11	·10	·09	·08	·07	·07	1·8	·99	·57	·34	..	..	..	..	..
,,	·300	·27	·21	·18	·15	·13	·12	·11	·10	·09	·08	2·1	1·2	·67	·40	..	..	..	..	..
2 × 2	·175	·21	·17	·14	·12	·11	·09	·08	·08	·07	·06	1·8	1·0	·63	·39	·26	..	..	..	..
,,	$\frac{1}{4}$	·29	·24	·20	·17	·15	·13	·12	·11	·10	·09	2·5	1·4	·88	·54	·36	..	..	..	..
,,	·300	·35	·28	·23	·20	·17	·15	·14	·13	·12	·11	3·0	1·6	1·2	·65	·42	..	..	..	..
$2\frac{1}{4} \times 2\frac{1}{4}$	·175	·27	·22	·18	·15	·14	·12	·11	·10	·09	·08	2·4	1·3	·88	·57	·37	·28	..	..	..
,,	$\frac{1}{4}$	·38	·30	·25	·21	·19	·17	·15	·14	·13	·12	3·3	1·9	1·2	·80	·52	·39	..	..	..
,,	·300	·45	·36	·30	·26	·23	·20	·18	·16	·15	·14	3·9	2·2	1·5	·94	·62	·46	..	..	..
$2\frac{1}{2} \times 2\frac{1}{2}$	$\frac{1}{4}$	·47	·38	·31	·27	·24	·21	·19	·17	·16	·15	4·1	2·4	1·6	1·1	·74	·50	·42	..	..
,,	$\frac{3}{8}$	·69	·55	·46	·39	·34	·31	·28	·25	·23	·21	6·0	3·5	2·3	1·6	1·1	·74	·62	..	..
,,	$\frac{1}{2}$	·89	·72	·60	·51	·45	·40	·36	·33	·30	·28	7·7	4·6	3·0	2·1	1·4	·96	·80	..	..
$2\frac{3}{4} \times 2\frac{3}{4}$	$\frac{1}{4}$	·58	·46	·39	·33	·29	·26	·23	·21	·19	·18	5·0	3·1	2·0	1·5	1·0	·73	·53	..	..
,,	$\frac{3}{8}$	·85	·68	·56	·48	·42	·38	·34	·31	·28	·26	7·3	4·6	3·0	2·1	1·5	1·1	·77	..	..
,,	$\frac{1}{2}$	1·1	·88	·73	·63	·55	·49	·44	·40	·37	·34	9·5	6·0	3·9	2·8	2·0	1·4	1·0	..	..
3 × 3	$\frac{1}{4}$	·69	·56	·46	·40	·35	·31	·28	·25	·23	·21	5·8	3·9	2·6	1·9	1·4	·98	·72	·56	..
,,	$\frac{3}{8}$	1·0	·81	·68	·58	·51	·45	·41	·37	·34	·31	8·5	5·7	3·8	2·7	2·0	1·4	1·1	·82	..
,,	$\frac{1}{2}$	1·3	1·1	·88	·76	·66	·59	·53	·48	·44	·41	11·1	7·5	4·9	3·6	2·6	1·9	1·4	1·1	..
$3\frac{1}{2} \times 3\frac{1}{2}$	·300	1·1	·91	·76	·65	·57	·50	·45	·41	·38	·35	8·7	6·5	4·6	3·3	2·5	1·9	1·4	1·2	·85
,,	$\frac{3}{8}$	1·4	1·1	·93	·80	·70	·62	·56	·51	·47	·43	11	8·0	5·6	4·1	3·1	2·4	1·8	1·4	1·1
,,	·425	1·6	1·3	1·1	·90	·79	·70	·63	·57	·53	·49	12	9·0	6·4	4·6	3·5	2·7	2·0	1·6	1·2
,,	$\frac{1}{2}$	1·8	1·5	1·2	1·1	·92	·82	·73	·67	·61	·57	15	11	8·0	5·7	4·4	3·4	2·5	2·0	1·5
4 × 4	·300	1·5	1·2	1·0	·85	·75	·66	·60	·54	·50	·46	11	9	6·2	4·5	3·4	2·7	2·2	1·7	1·3
,,	$\frac{3}{8}$	1·9	1·5	1·2	1·1	·93	·82	·74	·67	·62	·57	13	11	7·7	5·6	4·2	3·4	2·7	2·1	1·7
,,	·425	2·1	1·7	1·4	1·2	1·0	·92	·83	·76	·69	·64	15	12	8·6	6·3	4·7	3·8	3·0	2·3	1·9
,,	$\frac{1}{2}$	2·4	1·9	1·6	1·4	1·2	1·1	·97	·88	·81	·75	17	14	10	7·3	5·5	4·5	3·5	2·7	2·2
$4\frac{1}{2} \times 4\frac{1}{2}$	$\frac{3}{8}$	2·4	1·9	1·6	1·4	1·2	1·1	·94	·86	·79	·73	15	13	10	7·7	5·9	4·7	3·9	3·1	2·5
,,	$\frac{1}{2}$	3·1	2·5	2·1	1·8	1·6	1·4	1·2	1·1	1·0	·95	20	17	13	10	7·7	6·2	5·1	4·1	3·3
5 × 5	$\frac{3}{8}$	2·9	2·3	2·0	1·7	1·5	1·3	1·2	1·1	·98	·90	17	15	12	9·6	7·3	5·9	4·8	4·0	3·3
,,	$\frac{1}{2}$	3·9	3·1	2·6	2·2	1·9	1·7	1·5	1·4	1·3	1·2	23	20	16	13	9·6	7·7	6·4	5·3	4·4
6 × 6	·450	5·1	4·1	3·4	2·9	2·6	2·3	2·0	1·9	1·7	1·6	26	24	21	17	14	11	9·0	7·5	6·5
,,	$\frac{1}{2}$	5·6	4·5	3·8	3·2	2·8	2·5	2·3	2·0	1·9	1·7	29	26	23	19	15	12	9·9	8·3	7·2
,,	$\frac{5}{8}$	7·0	5·6	4·6	4·0	3·5	3·1	2·8	2·5	2·3	2·1	36	32	28	23	19	15	12	10	8·9
7 × 7	$\frac{1}{2}$	7·7	6·2	5·1	4·4	3·9	3·4	3·1	2·8	2·6	2·4	35	32	29	26	21	18	15	12	10
,,	$\frac{5}{8}$	9·5	7·6	6·4	5·5	4·8	4·2	3·8	3·5	3·2	2·9	43	39	36	32	26	22	18	15	13
,,	·675	10	8·2	6·9	5·9	5·1	4·6	4·1	3·7	3·4	3·2	46	42	38	34	28	24	20	16	14
8 × 8	·55	11	8·9	7·4	6·4	5·6	4·9	4·4	4·0	3·7	3·4	44	42	39	36	32	27	24	20	17
,,	$\frac{5}{8}$	13	10	8·4	7·2	6·3	5·6	5·0	4·6	4·2	3·9	50	47	44	40	36	31	27	23	19
,,	$\frac{3}{4}$	15	12	10	8·6	7·5	6·7	6·0	5·5	5·0	4·6	59	56	53	48	43	37	32	27	23

NOTES TO THE ABOVE TABLES.—CONTINUED.

nevertheless be used by adding one of the supplementary code words given in § 9, page 221 ("Code"). *E.g.* "FLOWER WHISKY" would denote *Rolled Steel Equal-sided Angle(s)* 5″ × 5″ × $\frac{3}{8}$″.

(5) The "Safe Distributed Loads" in the above table are calculated for a working stress of $7\frac{1}{2}$ tons per square inch by the formula:—Safe Load = 8 × Section Modulus (xx) × Working Stress ÷ Span (inches).

(6) The "Safe Loads as Struts" are calculated by the same formula as Table A of Safe Loads on Broad Flange Beams, page 157 ("Fidler's," Factor of Safety 5, "Fixed Ends"), with respect to the least radius of gyration (Axis vv in key drawing overleaf).

N.B.—Very small sizes of angles are used in a number of minor industries, down to about $\frac{1}{2}$″ × $\frac{1}{2}$″ × $\frac{1}{8}$″.

EQUAL-SIDED ANGLES.

METRIC STANDARD SECTIONS.

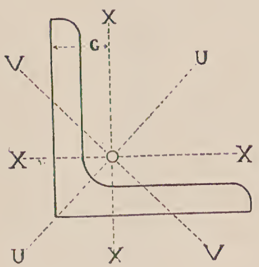
ENGLISH DIMENSIONS.			Reference No. and Code Word.		METRIC DIMENSIONS.					
Size. A × B	Thickness.	Weight per foot.			Size. A × B	Thickness.	Weight per metre.	Sectional Area.	Radii. C_1	Radii. C_2
Inches.	Ins.	Lbs.			Mm.	Mm.	Kilos.	Cm^2	Mm.	Mm.
0·59 × 0·59	0·12	0·43	702	Gable	15 × 15	3	0·64	0·82	3·5	2
,,	0·16	0·55	703	Gallant	,,	4	0·82	1·05	...	...
0·79 × 0·79	0·12	0·58	705	Gallop	20 × 20	3	0·87	1·12	3·5	2
,,	0·16	0·76	706	Gallows	,,	4	1·13	1·45	...	...
0·98 × 0·98	0·12	0·75	708	Gambler	25 × 25	3	1·11	1·42	3·5	2
,,	0·16	0·97	709	Gangway	,,	4	1·44	1·85	...	...
1·18 × 1·18	0·16	1·19	711	Garbage	30 × 30	4	1·77	2·27	5	2·5
,,	0·24	1·71	712	Garble	,,	6	2·55	3·27	...	...
1·38 × 1·38	0·16	1·40	714	Garden	35 × 35	4	2·08	2·67	5	2·5
,,	0·24	2·03	715	Garland	,,	6	3·02	3·87	...	...
1·57 × 1·57	0·16	1·61	717	Garment	40 × 40	4	2·40	3·08	6	3
,,	0·24	2·35	718	Garret	,,	6	3·49	4·48	...	...
,,	0·31	3·04	720	Garter	,,	8	4·52	5·80	...	...
1·77 × 1·77	0·20	2·26	721	Gauntlet	45 × 45	5	3·36	4·30	7	3·5
,,	0·28	3·07	723	General	,,	7	4·57	5·86	...	...
,,	0·35	3·85	724	Genius	,,	9	5·73	7·34	...	...
1·97 × 1·97	0·20	2·52	726	Gentile	50 × 50	5	3·75	4·80	7	3·5
,,	0·28	3·44	727	Gentry	,,	7	5·12	6·56	...	...
,,	0·35	4·31	729	Geology	,,	9	6·43	8·24	...	...
2·17 × 2·17	0·24	3·31	730	Geometry	55 × 55	6	4·92	6·31	8	4
,,	0·31	4·31	732	Gesture	,,	8	6·42	8·23	...	...
,,	0·39	5·27	733	Geyser	,,	10	7·85	10·1	...	...
2·36 × 2·36	0·24	3·62	735	Giant	60 × 60	6	5·39	6·91	8	4
,,	0·31	4·73	736	Gibbet	,,	8	7·04	9·03	...	...
,,	0·39	5·80	738	Gimlet	,,	10	8·63	11·1	...	...
2·56 × 2·56	0·28	4·56	739	Ginger	65 × 65	7	6·79	8·70	9	4·5
,,	0·35	5·75	741	Giraffe	,,	9	8·56	11·0	...	...
,,	0·43	6·92	742	Glacier	,,	11	10·3	13·2	...	...
2·76 × 2·76	0·28	4·93	744	Glebe	70 × 70	7	7·33	9·4	9	4·5
,,	0·35	6·22	745	Glimmer	,,	9	9·26	11·9	...	...
,,	0·43	7·46	747	Glimpse	,,	11	11·1	14·3	...	...
2·95 × 2·95	0·31	6·01	748	Gloom	75 × 75	8	8·94	11·5	10	5
,,	0·39	7·39	750	Glowworm	,,	10	11·0	14·1	...	...
,,	0·47	8·73	751	Glutton	,,	12	13·0	16·7	...	...
3·15 × 3·15	0·31	6·43	753	Goblet	80 × 80	8	9·57	12·3	10	5
,,	0·39	7·93	754	Goddess	,,	10	11·8	15·1	...	...
,,	0·47	9·34	756	Golfer	,,	12	13·9	17·9	...	...
3·54 × 3·54	0·35	8·13	757	Gondola	90 × 90	9	12·1	15·5	11	5·5
,,	0·43	9·81	759	Gosling	,,	11	14·6	18·7	...	...
,,	0·51	11·4	760	Gossip	,,	13	17·0	21·8	...	...

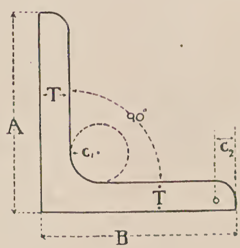
EQUAL-SIDED ANGLES.

METRIC STANDARD SECTIONS.—Continued.

ENGLISH DIMENSIONS.				Reference No. and Code Word.		METRIC DIMENSIONS.						
Size.		Thick-ness.	Weight per foot.			Size.		Thick-ness.	Weight per metre.	Sectional Area.	Radii.	
A	B					A	B				C_1	C_2
Inches.		Ins.	Lbs.			Mm.		Mm.	Kilos.	Cm^2	Mm.	Mm.
3·94 × 3·94		0·39	10·0	761	Gourmet	100 × 100		10	14·9	19·2	12	6
,,		0·47	11·9	763	Governor	,,		12	17·7	22·7	...	...
,,		0·55	13·7	764	Grammar	,,		14	20·4	26·2	...	...
4·33 × 4·33		0·39	11·1	766	Grandson	110 × 110		10	16·5	21·2	12	6
,,		0·47	13·2	767	Grange	,,		12	19·6	25·1	...	...
,,		0·55	15·2	769	Granite	,,		14	22·6	29·0	...	...
4·72 × 4·72		0·43	13·3	770	Grapnel	120 × 120		11	19·8	25·4	13	6·5
,,		0·51	15·6	772	Gravel	,,		13	23·2	29·7	...	...
,,		0·59	17·8	773	Gravy	,,		15	26·5	33·9	...	...
5·12 × 5·12		0·47	15·7	775	Grease	130 × 130		12	23·4	30·0	14	7
,,		0·55	18·1	776	Grenade	,,		14	27·0	34·7	...	...
,,		0·63	20·6	778	Grisette	,,		16	30·6	39·3	...	...
5·51 × 5·51		0·51	18·3	779	Gristle	140 × 140		13	27·3	35·0	15	7·5
,,		0·59	21·0	781	Groat	,,		15	31·2	40·0	...	...
,,		0·67	23·6	782	Grocer	,,		17	35·1	45·0	...	...
5·91 × 5·91		0·55	21·1	784	Groin	150 × 150		14	31·4	40·3	16	8
,,		0·63	24·0	785	Groom	,,		16	35·7	45·7	...	...
,,		0·71	26·8	787	Grouse	,,		18	39·9	51·0	...	...
6·30 × 6·30		0·59	24·1	788	Grove	160 × 160		15	35·9	46·1	17	8·5
,,		0·67	27·1	790	Growl	,,		17	40·4	51·8	...	...
,,		0·75	30·2	791	Guardian	,,		19	44·9	57·5	...	...

All of the above sizes from 45 × 45 mm. upwards are freely obtainable.





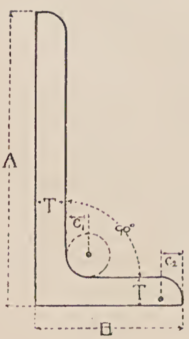
Misc. Shapes.
Misc. Tables.
Prices etc.
Code.
Grey Mill.
Tests etc.
R.I.B.A.
Photos.

BRITISH STANDARD UNEQUAL-SIDED ANGLES.

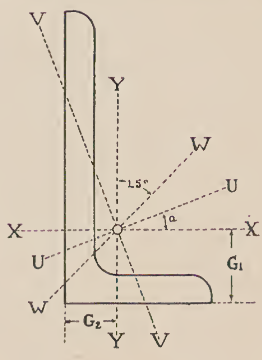
LIST OF SIZES AND PROPERTIES.

[For Safe Loads, P.T.O.]

Size. A × B	Thick-ness.		Weight per foot.	Reference No. and Code Word.		Sectional Area.	Radii. C1	Radii. C2	Moments of Inertia. XX	Moments of Inertia. YY	Moments of Inertia. UU	Moments of Inertia. VV	Section Moduli. XX	Section Moduli. YY
Inches.	Ins.		Lbs.			Sq. Ins.	Ins.	Ins.			Max.	Min.		
$1\frac{1}{4} \times 1$	$\frac{1}{8}$	c	·90	802	Habit	·265	·175	·125	·038	·022	·049	·011	·044	·030
,,	$\frac{1}{4}$		1·70	803	Haggis	·500	..	..	..	..	..	..	..	..
$1\frac{1}{2} \times 1\frac{1}{4}$	$\frac{1}{8}$	c	1·11	804	Hailstorm	·327	·200	·150	·068	·043	·090	·021	·064	·046
,,	$\frac{1}{4}$		2·12	805	Halibut	·624	..	..	..	..	..	..	..	..
$1\frac{3}{4} \times 1\frac{1}{2}$	·175	c	1·83	806	Hamlet	·539	·225	·150	·154	·104	·207	·051	·125	·095
,,	$\frac{1}{4}$*		2·55	807	Hammer	..	..	..	..	..	..	..	..	..
,,	·300		3·01	808	Hamper	·886	..	..	..	..	..	..	..	..
$2 \times 1\frac{1}{2}$	·175	a	1·98	810	Handcuff	·583	·225	·150	·225	·108	·275	·058	·163	·096
,,	$\frac{1}{4}$*		2·76	811	Handle	..	..	..	..	..	..	..	..	..
,,	·300		3·27	812	Harbinger	·961	..	..	..	..	..	..	..	..
$2\frac{1}{2} \times 2$	·175	c	2·57	814	Hardship	·757	·250	·175	·460	·260	·586	·134	·262	·173
,,	$\frac{1}{4}$*		3·61	815	Harmony	..	..	..	..	..	..	..	..	..
,,	·300		4·28	816	Harness	1·26	..	..	..	..	..	..	..	..
3×2	$\frac{1}{4}$	a	4·04	818	Harpoon	1·19	·275	·200	1·06	·373	1·21	·216	·522	·245
,,	$\frac{3}{8}$		5·89	819	Harpy	1·73	..	..	..	..	..	..	..	..
,,	$\frac{1}{2}$		7·65	820	Harrier	2·25	..	..	..	..	..	..	..	..
$3 \times 2\frac{1}{2}$	$\frac{1}{4}$	c	4·46	821	Harrow	1·31	·275	·200	1·14	·716	1·50	·356	·541	·387
,,	$\frac{3}{8}$		6·53	822	Harvest	1·92	..	..	..	..	..	..	..	..
,,	$\frac{1}{2}$		8·50	823	Hassock	2·50	..	..	..	..	..	..	..	..
$3\frac{1}{2} \times 2\frac{1}{2}$	$\frac{1}{4}$	c	4·90	824	Hatchet	1·44	·300	·200	1·76	·748	2·09	·415	·743	·394
,,	$\frac{3}{8}$		7·18	825	Haunch	2·11	..	..	..	..	..	..	..	..
,,	$\frac{1}{2}$		9·36	826	Haven	2·75	..	..	..	..	..	..	..	..
$3\frac{1}{2} \times 3$	$\frac{1}{4}$	c	5·31	827	Hawker	1·56	·325	·225	1·85	1·25	2·50	·608	·745	·562
,,	$\frac{3}{8}$		7·81	828	Hazard	2·30	..	..	..	..	..	..	..	..
,,	$\frac{1}{2}$		10·2	829	Hazel	3·00	..	..	..	..	..	..	..	..
$4 \times 2\frac{1}{2}$	$\frac{1}{4}$	ac	5·31	830	Headland	1·56	·325	·225	2·54	·767	2·85	·455	·939	·396
,,	$\frac{3}{8}$		7·81	831	Health	2·30	..	..	..	..	..	..	..	..
,,	$\frac{1}{2}$		10·2	832	Heater	3·00	..	..	..	..	..	..	..	..
4×3	·300	a	6·84	834	Heathen	2·01	·325	·225	3·19	1·54	3·90	·826	1·15	·680
,,	$\frac{3}{8}$*		8·45	835	Heaver	..	..	..	..	..	..	..	..	..
,,	·425		9·50	836	Hedge	2·80	..	..	..	..	..	..	..	..
,,	$\frac{1}{2}$		11·1	837	Heiress	3·25	..	..	..	..	..	..	..	..
$4 \times 3\frac{1}{2}$	·300	b	7·34	839	Helmet	2·16	·350	·250	3·33	2·38	4·58	1·13	1·17	·918
,,	$\frac{3}{8}$*		9·08	840	Helot	..	..	..	..	..	..	..	..	..
,,	·425		10·2	841	Hencoop	3·01	..	..	..	..	..	..	..	..
,,	$\frac{1}{2}$		11·9	842	Herbage	3·50	..	..	..	..	..	..	..	..
$4\frac{1}{2} \times 3$	·300	c	7·34	844	Heredity	2·16	·350	·250	4·41	1·57	5·08	·905	1·44	·683
,,	$\frac{3}{8}$*		9·08	845	Heresy	..	..	..	..	..	..	..	..	..
,,	·425		10·2	846	Heretic	3·01	..	..	..	..	..	..	..	..
,,	$\frac{1}{2}$		11·9	847	Heritage	3·50	..	..	..	..	..	..	..	..



The above table is continued on opposite page. For Safe Loads, Radii of Gyration etc., see table overleaf.



BRITISH STANDARD UNEQUAL-SIDED ANGLES.

LIST OF SIZES AND PROPERTIES.—Continued.

[For Safe Loads, P.T.O.]

Size. A × B	Thick-ness.		Weight per foot.	Reference No. and Code Word.		Sectional Area.	Radii. C_1	Radii. C_2	Moments of Inertia. XX	YY	UU	VV	Section Moduli. XX	YY
Inches.	Ins.		Lbs.			Sq. Ins.	Ins.	Ins.			Max.	Min.		
4½ × 3½	·300	*c*	7·85	849	Heroine	2·31	·350	·250	4·64	2·46	5·82	1·29	1·48	·932
,,	⅜*		9·72	850	History	..	..	..	..	..	..	..	..	.
,,	·425		10·9	851	Hockey	3·22	..	..	..	..	..	..	..	..
,,	½		12·8	852	Holster	3·75	..	..	..	..	..	..	..	..
5 × 3	·300	*a*	7·85	854	Hopper	2·31	·350	·250	5·91	1·62	6·55	·973	1·76	·692
,,	⅜*		9·72	855	Horizon	..	..	..	..	..	..	..	..	..
,,	·425		10·9	856	Hornpipe	3·22	..	..	..	..	..	..	..	..
,,	½		12·8	857	Horror	3·75	..	..	..	..	..	..	..	..
5 × 3½	⅜	*ac*	10·4	859	Hostage	3·05	·375	·250	7·64	3·10	9·01	1·73	2·24	1·17
,,	½		13·6	860	Hostler	4·00	..	..	..	..	..	..	..	..
5 × 4	⅜	*ac*	11·0	862	Humanity	3·24	·400	·275	7·96	4·53	10·2	2·32	2·28	1·52
,,	½		14·5	863	Humility	4·25	..	..	..	..	..	..	..	..
5½ × 3	⅜	*c*	10·4	865	Hunger	3·05	·375	·250	9·45	2·02	10·2	1·26	2·62	·864
,,	½		13·6	866	Hussar	4·00	..	..	..	..	..	..	..	..
5½ × 3½	⅜	*c*	11·0	868	Hybrid	3·24	·400	·275	9·93	3·16	11·2	1·85	2·68	1·17
,,	½		14·5	869	Hygiene	4·25	..	..	..	..	..	..	..	..
6 × 3	⅜*	*d*	11·0	871	Hyperbole	..	..	..	..	..	..	..	..	..
,,	½*		14·5	872	Hypnotism	..	..	..	..	..	..	..	..	..
6 × 3½	⅜	*a*	11·6	873	Hypocrite	3·42	·400	·275	12·6	3·23	13·9	1·96	3·17	1·18
,,	½		15·3	874	Hyssop	4·50	..	..	..	..	..	..	..	..
,,	⅝*		18·9	875	Hysterics	..	..	..	..	..	..	..	..	..
6 × 4	⅜	*a*	12·3	876	Iceberg	3·61	·425	·300	13·2	4·73	15·2	2·71	3·23	1·54
,,	½		16·2	877	Icicle	4·75	..	..	..	..	..	..	..	..
,,	⅝*		19·9	878	Ideal	..	..	..	..	..	..	..	..	..
,,	¾*		23·6	879	Identity	..	..	..	..	..	..	..	..	..
6½ × 3½	⅜	*c*	12·3	880	Idiom	3·61	·425	·300	15·7	3·27	16·9	2·05	3·68	1·18
,,	½		16·2	881	Idler	4·75	..	..	..	..	..	..	..	..
6½ × 4	·525	*c*	17·8	883	Ignition	5·24	·425	·300	22·4	6·50	25·0	3·89	5·19	2·13
6½ × 4½	·550	*c*	19·5	885	Imagery	5·75	·450	·325	24·2	9·51	28·4	5·37	5·51	2·81
7 × 3½	·525	*b*	17·8	887	Imitation	5·24	·425	·300	26·2	4·46	27·8	2·88	5·84	1·64
7 × 4	·550	*b*	19·5	888	Impostor	5·75	·450	·325	28·6	6·86	31·2	4·23	6·23	2·23
,,	⅝*		22·1	889	Impulse	..	..	..	..	..	..	..	..	..
,,	¾*		26·1	890	Inanity	..	..	..	..	..	..	..	..	..
8 × 3½	·575	*c*	21·4	891	Incline	6·29	·475	·325	41·1	4·93	42·7	3·32	8·19	1·79
8 × 4	⅝	*d*	24·2	893	Increment	7·11	·475	·325	46·4	7·89	49·2	5·11	9·08	2·54
9 × 4	⅝*	*a*	26·3	895	Indecorum	..	..	..	..	..	..	..	..	..
,,	·650		27·3	896	Indianum	8·03	·500	·350	66·6	8·34	69·3	5·58	11·8	2·66
,,	¾*		31·2	897	Indigence	..	..	..	..	..	..	..	..	..
10 × 4	·675	*d*	30·6	898	Indignity	9·00	·550	·375	92·1	8·77	94:8	6·03	14·9	2·77

NOTES TO THE ABOVE TABLE.

(1) *Sizes marked* "*a*" are promptly obtainable in any quantities and freely stocked.
Sizes marked "*b*" are in common demand and frequently rolled.
Sizes marked "*c*" should be avoided unless 50 tons of a section are required (or say 20 tons in the case of sections weighing 8 lbs. per foot or less).
Sizes marked "*ac*" are stocked in limited quantities but are not rolled regularly and frequently.
Sizes marked "*d*":—6″×3″ angles are not standard but are in common use and freely stocked at present; 8″×4″ angles are not readily obtainable from British works at present, but the size is a convenient one in structural work and a closely approximate metric section (7⅞″×3$\frac{15}{16}$″, No. 981, page 204) is stocked by girder merchants in London and elsewhere.

(2) The odd thicknesses given in decimals of an inch, although standard, are intended mainly for shipbuilding and are preferably avoided by the ordinary user.

(3) Thicknesses marked with an asterisk (*) are not official standard thicknesses.

(4) By lifting the rolls, nearly all of the above sizes can be rolled to greater thicknesses than those given in the table. All intermediate thicknesses can be supplied in like fashion.

(5) In the case of thicknesses not given in the table, the above code words can be used nevertheless, by adding one of the supplementary code words given in § 9, page 221 ("Code").

(6) The largest unequal angle rolled is 12″×10″ by ⅞″ and 1″. 11″×4″ angles ⅜″ to ¾″ thick are rolled by three makers. But these exceptional sizes should be avoided unless about 100 tons of a section are required.

BRITISH STANDARD UNEQUAL-SIDED ANGLES.

TABLE OF SAFE LOADS.

[For Properties, Code Words Etc., see previous Table, page 200.]

Size. A × B	Thickness.		SAFE DISTRIBUTED LOADS (TONS) AT 7½ TONS STRESS. Long Leg Vertical. 4′	5′	6′	7′	8′	9′	Short Leg Vertical. 4′	5′	6′	7′	8′	9′	Radii of Gyration. XX	YY	UU Max.	VV Min.	Reference No.
Inches.	Ins.																		
1¼ × 1	⅛	c	·05	·04	·04	·03	·03	·02	·04	·03	·03	·02	·02	·02	·384	·288	·430	·204	802
,,	¼		·10	·08	·07	·06	·05	·05	·07	·06	·05	·04	·04	·03	..	..	..	..	803
1½ × 1¼	⅛	c	·08	·06	·05	·05	·04	·04	·06	·05	·04	·03	·03	·03	·456	·363	·525	·253	804
,,	¼		·15	·12	·10	·09	·07	·07	·11	·09	·07	·06	·05	·05	..	..	..	..	805
1¾ × 1½	·175	c	·15	·13	·10	·09	·08	·07	·12	·10	·08	·07	·06	·05	·535	·439	·620	·308	806
,,	¼		·22	·17	·15	·12	·11	·10	·16	·13	·11	·09	·08	·07	..	..	..	..	807
,,	·300		·26	·21	·17	·15	·13	·11	·19	·16	·13	·11	·10	·09	..	..	..	..	808
2 × 1½	·175	a	·20	·16	·14	·12	·10	·09	·12	·10	·08	·07	·06	·05	·621	·430	·687	·315	810
,,	¼		·28	·23	·19	·16	·14	·13	·17	·13	·11	·10	·08	·07	..	..	..	..	811
,,	·300		·34	·27	·22	·19	·17	·15	·20	·16	·13	·11	·10	·09	..	..	..	..	812
2½ × 2	·175	c	·33	·26	·22	·19	·16	·15	·22	·17	·14	·12	·10	·10	·780	·586	·880	·421	814
,,	¼		·46	·37	·31	·26	·23	·20	·30	·24	·20	·17	·15	·14	..	..	..	..	815
,,	·300		·54	·44	·36	·31	·27	·24	·36	·29	·24	·21	·18	·16	..	..	..	..	816
3 × 2	¼	a	·65	·52	·44	·37	·33	·29	·31	.25	·20	·18	·15	·14	·943	·561	1·01	·427	818
,,	⅜		·95	·76	·63	·54	·48	·42	·45	·36	·30	·26	·22	·20	..	..	..	..	819
,,	½		1·2	·99	·82	·71	·62	·55	·58	·46	·39	·33	·29	·26	..	..	..	..	820
3 × 2½	¼	c	·68	·54	·45	·39	·34	·30	·48	·39	·32	·28	·24	·21	·931	·739	1·07	·521	821
,,	⅜		·99	·79	·66	·57	·50	·44	·71	·57	·47	·40	·35	·31	..	..	..	..	822
,,	½		1·3	1·0	·86	·74	·64	·57	·92	·74	·61	·53	·46	·41	..	..	..	..	823
3½ × 2½	¼	c	·93	·74	·62	·53	·46	·41	·49	·39	·33	·28	·25	·22	1·11	·721	1·21	·537	824
,,	⅜		1·4	1·1	·91	·78	·68	·61	·72	·58	·48	·41	·36	·32	..	..	..	..	825
,,	½		1·8	1·4	1·2	1·0	·89	·79	·94	·75	·63	·54	·47	·42	..	..	..	..	826
3½ × 3	¼	c	·93	·75	·62	·53	·47	·41	·70	·56	·47	·40	·35	·31	1·09	·896	1·26	·624	827
,,	⅜		1·4	1·1	·91	·78	·68	·61	1·0	·83	·69	·59	·52	·46	..	..	..	..	828
,,	½		1·8	1·4	1·2	1·0	·89	·79	1·3	1·1	·90	·77	·67	·60	..	..	..	..	829
4 × 2½	¼	ac	1·2	·94	·78	·67	·59	·52	·49	·40	·33	·28	·25	·22	1·27	·701	1·35	·540	830
,,	⅜		1·7	1·4	1·1	·99	·86	·77	·73	·58	·49	·42	·36	·32	..	..	..	..	831
,,	½		2·2	1·8	1·5	1·3	1·1	1·0	·95	·76	·63	·54	·48	·42	..	..	..	..	832
4 × 3	·300	a	1·4	1·2	·96	·82	·72	·64	·85	·68	·57	·49	·43	·38	1·26	·874	1·39	·641	834
,,	⅜		1·8	1·4	1·2	1·0	·89	·79	1·1	·84	·70	·60	·53	·47	..	..	..	..	835
,,	·425		2·0	1·6	1·3	1·1	1·0	·89	1·2	·95	·79	·68	·59	·52	..	..	..	..	836
,,	½		2·3	1·9	1·6	1·3	1·2	1·0	1·4	1·1	·92	·79	·69	·61	..	..	..	..	837
4 × 3½	·300	b	1·5	1·2	·98	·84	·73	·65	1·1	·92	·77	·66	·57	·51	1·24	1·05	1·46	·723	839
,,	⅜		1·8	1·5	1·2	1·0	·91	·81	1·4	1·1	·95	·81	·71	·63	..	..	..	..	840
,,	·425		2·0	1·6	1·4	1·2	1·0	·91	1·6	1·3	1·1	·91	·80	·71	..	..	..	..	841
,,	½		2·4	1·9	1·6	1·4	1·2	1·1	1·9	1·5	1·2	1·1	·93	·83	..	..	..	..	842
4½ × 3	·300	c	1·8	1·4	1·2	1·0	·90	·80	·85	·68	·57	·49	·43	·38	1·43	·854	1·53	·647	844
,,	⅜		2·2	1·8	1·5	1·3	1·1	·99	1·1	·84	·70	·60	·53	·47	..	..	..	..	845
,,	·425		2·5	2·0	1·7	1·4	1·3	1·1	1·2	·95	·79	·68	·59	·53	..	..	..	..	846
,,	½		2·9	2·3	1·9	1·7	1·5	1·3	1·4	1·1	·92	·79	·69	·62	..	..	..	..	847

BRITISH STANDARD UNEQUAL-SIDED ANGLES.

TABLE OF SAFE LOADS.—Continued.

[For Properties, Code Words Etc., see previous Table, page 200.]

Size. A × B	Thickness.		SAFE DISTRIBUTED LOADS (TONS) AT 7½ TONS STRESS. Long Leg Vertical. 4′	5′	6′	7′	8′	9′	Short Leg Vertical. 4′	5′	6′	7′	8′	9′	Radii of Gyration. XX	YY	UU	VV	Reference No.
Inches.	Ins.																Max.	Min	
4½ × 3½	·300	c	1·8	1·5	1·2	1·1	·92	·82	1·2	·93	·78	·67	·58	·52	1·42	1·03	1·59	·746	849
,,	⅜		2·3	1·8	1·5	1·3	1·1	1·0	1·4	1·2	·96	·82	·72	·64	..	..	..	..	850
,,	·425		2·6	2·1	1·7	1·5	1·3	1·1	1·6	1·3	1·1	·93	·81	·72	..	..	..	..	851
,,	½		3·0	2·4	2·0	1·7	1·5	1·3	1·9	1·5	1·3	1·1	·95	·84	..	..	..	..	852
5 × 3	·300	a	2·2	1·8	1·5	1·3	1·1	·98	·87	·69	·58	·49	·43	·38	1·60	·837	1·69	·649	854
,,	⅜		2·7	2·2	1·8	1·6	1·4	1·2	1·1	·86	·71	·61	·54	·48	..	..	..	..	855
,,	·425		3·1	2·5	2·1	1·8	1·5	1·4	1·2	·96	·80	·69	·60	·54	..	..	..	..	856
,,	½		3·6	2·9	2·4	2·0	1·8	1·6	1·4	1·1	·94	·80	·70	·63	..	..	..	..	857
5 × 3½	⅜	ac	2·8	2·2	1·9	1·6	1·4	1·2	1·5	1·2	·97	·83	·73	·64	1·58	1·01	1·72	·754	859
,,	½		3·7	2·9	2·5	2·1	1·8	1·6	1·9	1·5	1·3	1·1	·96	·85	..	..	..	..	860
5 × 4	⅜	ac	2·8	2·3	1·9	1·6	1·4	1·3	1·9	1·5	1·3	1·1	·95	·84	1·57	1·18	1·77	·847	862
,,	½		3·7	3·0	2·5	2·1	1·9	1·7	2·5	2·0	1·7	1·4	1·2	1·1	..	..	..	..	863
5½ × 3	⅜	c	3·3	2·6	2·2	1·9	1·6	1·5	1·1	·86	·72	·62	·54	·48	1·76	·814	1·83	·644	865
,,	½		4·3	3·4	2·9	2·5	2·2	1·9	1·4	1·1	·95	·81	·71	·63	..	..	..	..	866
5½ × 3½	⅜	c	3·4	2·7	2·2	1·9	1·7	1·5	1·5	1·2	·98	·84	·73	·65	1·75	·987	1·86	·757	868
,,	½		4·4	3·5	2·9	2·5	2·2	2·0	1·9	1·5	1·3	1·1	·96	·86	..	..	..	..	869
6 × 3½	⅜	a	4·0	3·2	2·6	2·3	2·0	1·8	1·5	1·2	·99	·85	·74	·66	1·92	·971	2·02	·757	873
,,	½		5·2	4·2	3·5	3·0	2·6	2·3	1·9	1·6	1·3	1·1	·97	·86	..	..	..	..	874
6 × 4	⅜	a	4·0	3·2	2·7	2·3	2·0	1·8	1·9	1·5	1·3	1·1	·96	·85	1·91	1·15	2·05	·867	876
,,	½		5·3	4·2	3·5	3·0	2·7	2·4	2·5	2·0	1·7	1·4	1·3	1·1	..	..	..	..	877
6½ × 3½	⅜	c	4·6	3·7	3·1	2·6	2·3	2·0	1·5	1·2	·99	·85	·74	·66	2·09	·951	2·17	·753	880
,,	½		6·1	4·8	4·0	3·5	3·0	2·7	1·9	1·6	1·3	1·1	·97	·87	..	..	..	..	881
6½ × 4	·525	c	6·5	5·2	4·3	3·7	3·2	2·9	2·7	2·1	1·8	1·5	1·3	1·2	2·07	1·11	2·18	·862	883
6½ × 4½	·550	c	6·9	5·5	4·6	3·9	3·4	3·1	3·5	2·8	2·3	2·0	1·8	1·6	2·05	1·29	2·22	·967	885
7 × 3½	·525	b	7·3	5·8	4·9	4·2	3·7	3·2	2·0	1·6	1·4	1·2	1·0	·91	2·24	·922	2·30	·742	887
7 × 4	·550	b	7·8	6·2	5·2	4·4	3·9	3·5	2·8	2·2	1·9	1·6	1·4	1·2	2·23	1·09	2·33	·858	888
8 × 3½	·575	c	10	8·2	6·8	5·8	5·1	4·6	2·2	1·8	1·5	1·3	1·1	·99	2·56	·886	2·61	·727	891
8 × 4	⅝	d	11	9·1	7·6	6·5	5·7	5·0	3·2	2·5	2·1	1·8	1·6	1·4	2·56	1·05	2·63	·847	893
9 × 4	·650	a	15	12	9·8	8·4	7·4	6·5	3·3	2·7	2·2	1·9	1·7	1·5	2·88	1·02	2·94	·834	896
10 × 4	·675	d	19	15	12	11	9·3	8·3	3·5	2·8	2·3	2·0	1·7	1·5	3·20	·987	3·25	·819	898

NOTES TO THE ABOVE TABLE.

(1) For explanation of the letters *a*, *b* etc. in the "Thickness" column, see previous table, page 201.

(2) The safe distributed loads are calculated for a working stress of 7½ tons per square inch (ignoring shearing stress etc.) by the following formulæ for transverse bending :—

Safe Load with Long Leg Vertical = 8 × *Section Modulus* (XX) × *Working Stress* ÷ *Span* (*inches*).

Safe Load with Short Leg Vertical = 8 × *Section Modulus* (YY) × *Working Stress* ÷ *Span* (*inches*).

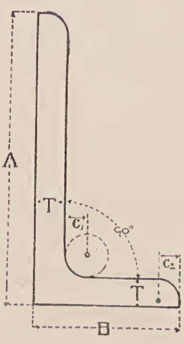
(3) Some of the above thicknesses are not standard (see previous table).

(4) The previous table ("Properties") includes some sizes not given above.

UNEQUAL-SIDED ANGLES.

METRIC STANDARD SECTIONS.

ENGLISH DIMENSIONS.			Reference No. and Code Word.		METRIC DIMENSIONS.					
Size. A × B	Thickness.	Weight per foot.			Size A × B	Thickness.	Weight per metre.	Sectional Area.	Radii. C1	C2
Inches.	Ins.	Lbs.			Mm.	Mm.	Kilos.	Cm²	Mm.	Mm.
1·18 × 0·79	·12	·75	903	Jacket	30 × 20	3	1·11	1·42	3·5	2
,,	·16	·97	906	Jacobite	,,	4	1·44	1·85	,,	,,
1·57 × 0·79	·12	·90	909	Jaguar	40 × 20	3	1·34	1·72	3·5	2
,,	·16	1·18	912	Jailer	,,	4	1·76	2·25	,,	,,
1·77 × 1·18	·16	1·51	915	Jargon	45 × 30	4	2·24	2·87	4·5	2
,,	·20	1·85	918	Jasmine	,,	5	2·75	3·53	,,	,,
2·36 × 1·18	·20	2·25	921	Jasper	60 × 30	5	3·35	4·29	6	3
,,	·28	3·07	924	Javelin	,,	7	4·56	5·85	,,	,,
2·36 × 1·57	·20	2·51	927	Jealousy	60 × 40	5	3·74	4·79	6	3
,,	·28	3·44	930	Jeopardy	,,	7	5·11	6·55	,,	,,
2·95 × 1·97	·28	4·37	933	Jerkin	75 × 50	7	6·50	8·33	8	4
,,	·35	5·51	936	Jersey	,,	9	8·20	10·5	,,	,,
3·15 × 1·57	·24	3·61	939	Jester	80 × 40	6	5·37	6·89	7	3·5
,,	·31	4·72	942	Jesuitism	,,	8	7·03	9·01	,,	,,
3·94 × 1·97	·31	6·00	945	Jewel	100 × 50	8	8·93	11·5	9	4·5
,,	·39	7·39	948	Jeweller	,,	10	11·0	14·1	,,	,,
3·94 × 2·56	·35	7·39	951	Jingo	100 × 65	9	11·0	14·2	10	5
,,	·43	8·93	954	Jobber	,,	11	13·3	17·1	,,	,,
4·72 × 3·15	·39	10·0	957	Jockey	120 × 80	10	14·9	19·1	11	5·5
,,	·47	11·9	960	Jointure	,,	12	17·7	22·7	,,	,,
5·12 × 2·56	·39	9·74	963	Jollity	130 × 65	10	14·5	18·6	11	5·5
,,	·47	11·6	966	Jotting	,,	12	17·2	22·1	,,	,,
5·91 × 3·94	·47	15·1	969	Joust	150 × 100	12	22·4	28·7	13	6·5
,,	·55	17·4	972	Joviality	,,	14	25·9	33·2	,,	,,
6·30 × 3·15	·47	14·4	975	Jubilee	160 × 80	12	21·5	27·5	13	6·5
,,	·55	16·8	978	Judaism	,,	14	24·8	31·8	,,	,,
7·87 × 3·94	·55	21·1	981	Judge	200 × 100	14	31·4	40·3	15	7·5
,,	·63	23·9	984	Juggler	,,	16	35·6	45·7	,,	,,

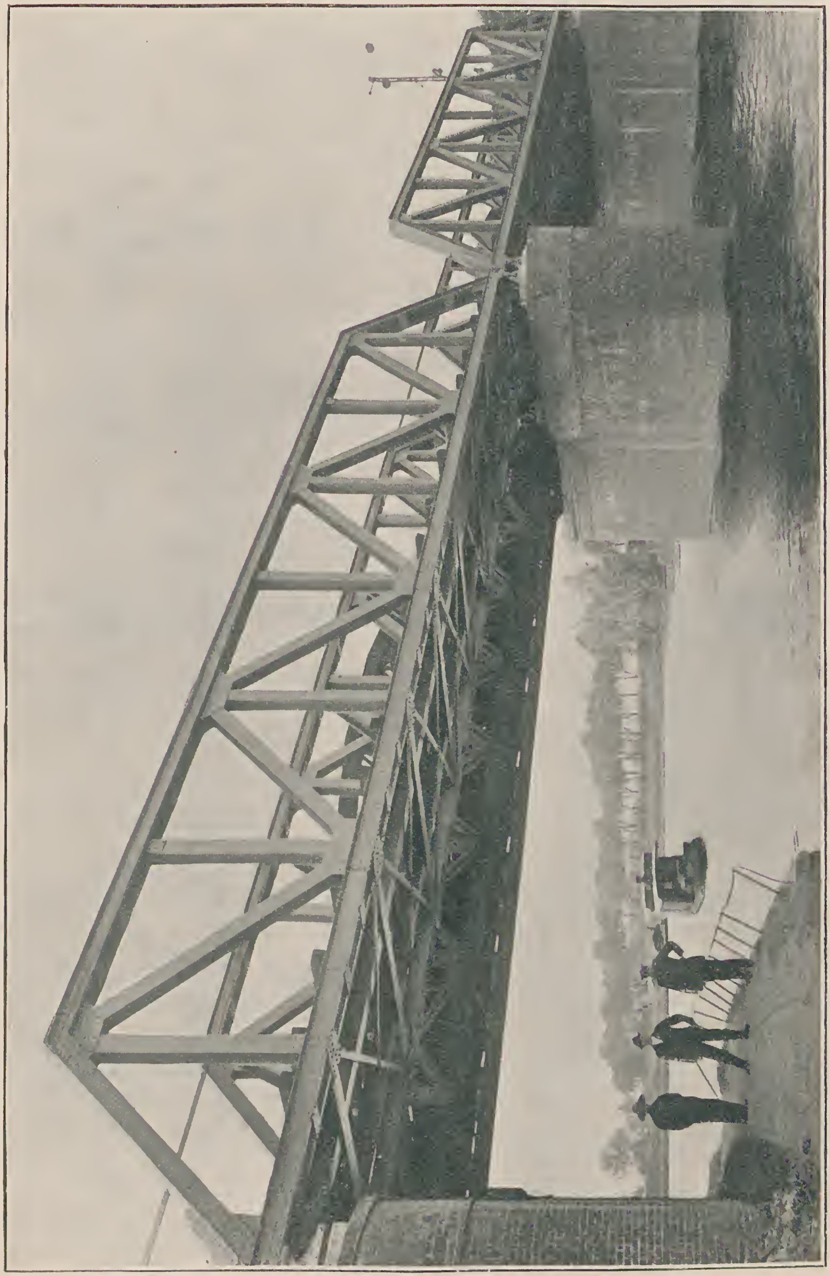


NOTE TO THE ABOVE TABLE.

The above are the German standard sections of unequal angles for general structural purposes. In addition to these there are about 50 standard sections for shipbuilding (not so readily obtainable), ranging from 40 × 30 mm. up to 250 × 90 mm.

BROAD FLANGE BEAMS IN RAILWAY BRIDGE CONSTRUCTION.

[For further examples, see pages 234 to 240.]



BRITISH STANDARD TEES.

LIST OF SIZES AND PROPERTIES.

[For Key Drawing, P.T.O.]

Size. A × B	Thickness.		Weight per foot.	Reference No. and Code Word.		Sectional Area.	Radii. C_1	Radii. C_2	Moments of Inertia. XX	Moments of Inertia. YY	Section Moduli. XX	Section Moduli. YY	G
Inches.	Ins.		Lbs.			Sq. Ins.	Ins.	Ins.					Ins.
1 × 1	$\frac{1}{8}$	*a*	·82	1002	Kaiser	·240	·175	·125	·021	·009	·030	·018	·289
,,	$\frac{3}{16}$	*a*	1·17	1004	Keenness	·344	...	...	...	...	...	...	...
$1\frac{1}{4} \times 1\frac{1}{4}$	$\frac{1}{8}$	*a*	1·03	1006	Keeper	·303	·200	·150	·042	·017	·047	·027	·348
,,	$\frac{3}{16}$	*a*	1·49	1008	Kennel	·438	...	...	...	...	...	...	...
$1\frac{1}{2} \times 1\frac{1}{2}$	$\frac{3}{16}$	*a*	1·81	1010	Kerchief	·531	·200	·150	·106	·048	·100	·064	·435
,,	$\frac{1}{4}$	*a*	2·35	1012	Kernel	·692	...	...	...	...	...	...	...
$1\frac{3}{4} \times 1\frac{3}{4}$	$\frac{3}{16}$	*a*	2·14	1014	Kersey	·629	·225	·150	·173	·077	·138	·088	·492
,,	$\frac{1}{4}$	*a*	2·79	1016	Kestrel	·820	...	...	...	...	...	...	...
$1\frac{1}{2} \times 2$	$\frac{1}{4}$	*c*	2·79	1018	Ketch	·820	·225	·150	·307	·068	·227	·091	·648
,,	$\frac{5}{16}$	*c*	3·40	1020	Ketchup	1·00	...	...	...	...	...	...	...
2 × 2	$\frac{1}{4}$	*a*	3·22	1022	Kettle	·947	·250	·175	·337	·157	·237	·157	·579
,,	$\frac{5}{16}$	*a*	3·94	1024	Keyhole	1·16	...	...	...	...	...	...	...
,,	$\frac{3}{8}$	*a*	4·64	1026	Keystone	1·37	...	...	...	...	...	...	...
$2\frac{1}{4} \times 2\frac{1}{4}$	$\frac{1}{4}$	*a*	3·64	1028	Kicker	1·07	·250	·175	·488	·224	·303	·199	·638
,,	$\frac{5}{16}$	*a*	4·47	1030	Kidnapper	1·31	...	...	...	...	...	...	...
,,	$\frac{3}{8}$	*a*	5·28	1032	Kidney	1·55	...	...	...	...	...	...	...
$2\frac{1}{2} \times 2\frac{1}{2}$	$\frac{1}{4}$	*a*	4·07	1034	Killer	1·20	·275	·200	·677	·302	·375	·242	·697
,,	$\frac{5}{16}$	*a*	5·00	1036	Kiln	1·47	...	...	...	...	...	...	...
,,	$\frac{3}{8}$	*a*	5·92	1038	Kindness	1·74	...	...	...	...	...	...	...
3 × 2	$\frac{5}{16}$	*b*	5·01	1040	Kindred	1·47	·275	·200	·457	·666	·307	·444	·509
,,	$\frac{3}{8}$	*b*	5·93	1042	Kingcraft	1·74	...	...	...	...	...	...	...
$3 \times 2\frac{1}{2}$	$\frac{5}{16}$	*b*	5·53	1044	Kingdom	1·63	·275	·200	·869	·667	·475	·445	·670
,,	$\frac{3}{8}$	*b*	6·56	1046	Kinsman	1·93	...	...	...	...	...	...	...
3 × 3	$\frac{5}{16}$	*a*	6·08	1048	Kinswomen	1·79	·300	·200	1·46	·669	·675	·446	·842
,,	$\frac{3}{8}$	*a*	7·21	1050	Kiosk	2·12	...	...	...	...	...	...	...
,,	$\frac{7}{16}$	*a*	8·30	1052	Kipper	2·44	...	...	...	...	...	...	...
3 × 4	$\frac{3}{8}$	*d*	8·48	1054	Kitchener	2·49	·325	·225	3·82	·812	1·39	·541	1·24
,,	$\frac{1}{2}$	*d*	11·1	1056	Kitten	3·26	...	...	...	...	...	...	...
$3\frac{1}{2} \times 3\frac{1}{2}$	$\frac{3}{8}$	*a*	8·49	1058	Knack	2·50	·325	·225	2·77	1·28	1·10	·734	·988
,,	$\frac{7}{16}$	*a*	9·78	1060	Knapsack	2·88	...	...	...	...	...	...	...
,,	$\frac{1}{2}$	*a*	11·1	1062	Knave	3·26	...	...	...	...	...	...	...
4 × 3	$\frac{3}{8}$	*a*	8·49	1064	Knavery	2·50	·325	·225	1·86	1·91	·833	·957	·767
,,	$\frac{1}{2}$	*a*	11·1	1066	Knavish	3·26	...	...	...	...	...	...	...
4 × 4	$\frac{3}{8}$	*a*	9·77	1068	Kneecap	2·87	·350	·250	4·19	1·90	1·45	·950	1·11
,,	$\frac{1}{2}$	*a*	12·8	1070	Knell	3·76	...	...	...	...	...	...	...
4 × 5	$\frac{3}{8}$	*b*	11·1	1072	Knitter	3·25	·400	·275	7·77	1·89	2·20	·943	1·47
,,	$\frac{1}{2}$	*b*	14·5	1074	Knives	4·26	...	...	...	...	...	...	...
5 × 3	$\frac{3}{8}$	*a*	9·78	1076	Knob	2·88	·350	·250	1·97	3·72	·854	1·49	·691
,,	$\frac{1}{2}$	*a*	12·8	1078	Knocker	3·76	...	...	...	...	...	...	...
$5 \times 3\frac{1}{2}$	$\frac{1}{2}$	*a*	13·7	1080	Knolls	4·02	·375	·250	3·94	5·04	1·51	2·02	·892
5 × 4	$\frac{1}{2}$	*a*	14·5	1082	Knots	4·27	·400	·275	5·77	5·02	1·96	2·01	1·05
6 × 3	$\frac{3}{8}$	*a*	11·1	1084	Knuckle	3·26	·400	·275	2·06	6·39	·871	2·13	·633
,,	$\frac{1}{2}$	*a*	14·5	1086	Koran	4·27	...	...	...	...	...	...	...
6 × 4	$\frac{1}{2}$	*a*	16·2	1088	Kraal	4·77	·425	·300	6·07	8·62	2·00	2·87	·968
$7 \times 3\frac{1}{2}$	$\frac{1}{2}$	*d*	17·1	1090	Kyaniser	5·02	·425	·300	4·29	13·7	1·57	3·91	·764

NOTES TO THE ABOVE TABLE.

(1) It is important to give the dimensions of Tees in the same order as above.
(2) *Sizes marked* "*a*" are frequently rolled and freely stocked.
Sizes marked "*b*" are frequently rolled
Sizes marked "*c*" are less readily obtainable.
Sizes marked "*d*" should be avoided unless about 50 tons of a size are required.

N.B.—Some further notes are given overleaf, with key drawing.

BRITISH STANDARD TEES.—Continued.

LIST OF SIZES, SAFE LOADS AND RADII OF GYRATION.

Size.			SAFE DISTRIBUTED LOADS (TONS).									SAFE LOADS AS STRUTS (TONS).								Radii of Gyration.	
A	B	T	5′	6′	7′	8′	9′	10′	11′	12′	13′	4′	6′	8′	10′	12′	14′	16′	18′	XX	YY
Inches.																				Ins.	Ins.
1	× 1	× 1/8	·03	…	…	…	…	…	…	…	…	·23	·09	…	…	…	…	…	…	·296	·194
,,		× 3/16	·04	…	…	…	…	…	…	…	…	·32	·13	…	…	…	…	…	…	…	…
1¼	× 1¼	× 1/8	·05	…	…	…	…	…	…	…	…	·40	·18	…	…	…	…	…	…	·372	·237
,,		× 3/16	·07	…	…	…	…	…	…	…	…	·57	·27	…	…	…	…	…	…	…	…
1½	× 1½	× 3/16	·10	·08	·07	…	…	…	…	…	…	·98	·53	·28	·19	…	…	…	…	·447	·301
,,		× ¼	·13	·11	·09	…	…	…	…	…	…	1·3	·69	·37	·25	…	…	…	…	…	…
1¾	× 1¾	× 3/16	·14	·12	·10	·09	·08	…	…	…	…	1·5	·81	·47	·27	…	…	…	…	·524	·350
,,		× ¼	·18	·15	·13	·11	·10	…	…	…	…	1·9	1·1	·62	·36	…	…	…	…	…	…
1½	× 2	× ¼	·23	·19	·16	·14	·13	·11	…	…	…	1·4	·75	·39	…	…	…	…	…	·612	·288
,,		× 5/16	·28	·23	·20	·17	·15	·14	…	…	…	1·7	·92	·47	…	…	…	…	…	…	…
2	× 2	× ¼	·24	·20	·17	·15	·13	·12	…	…	…	2·7	1·5	·98	·61	·39	…	…	…	·597	·407
,,		× 5/16	·29	·24	·21	·18	·16	·15	…	…	…	3·3	1·9	1·2	·74	·48	…	…	…	…	…
,,		× 3/8	·34	·28	·24	·21	·19	·17	…	…	…	3·9	2·2	1·4	·88	·57	…	…	…	…	…
2¼	× 2¼	× ¼	·30	·25	·22	·19	·17	·15	·14	…	…	3·5	2·0	1·3	·88	·59	·42	…	…	·675	·457
,,		× 5/16	·37	·31	·27	·23	·21	·19	·17	…	…	4·2	2·5	1·6	1·1	·73	·51	…	…	…	…
,,		× 3/8	·44	·37	·31	·27	·24	·22	·20	…	…	5·0	2·9	1·9	1·3	·86	·60	…	…	…	…
2½	× 2½	× ¼	·38	·32	·27	·23	·21	·19	·17	·16	…	4·3	2·6	1·7	1·2	·81	·56	·44	…	·752	·502
,,		× 5/16	·46	·38	·33	·29	·25	·23	·21	·19	…	5·2	3·2	2·1	1·5	1·0	·68	·54	…	…	…
,,		× 3/8	·55	·45	·39	·34	·30	·27	·25	·23	…	6·2	3·8	2·5	1·7	1·2	·81	·64	…	…	…
3	× 2	× 5/16	·31	·26	·22	·19	·17	·15	…	…	…	6·3	4·7	3·2	2·3	1·8	1·4	1·0	·78	·557	·673
,,		× 3/8	·36	·30	·26	·23	·20	·18	…	…	…	7·5	5·5	3·8	2·8	2·1	1·6	1·2	·92	…	…
3	× 2½	× 5/16	·48	·40	·34	·29	·26	·24	·22	·20	…	6·8	4·8	3·3	2·4	1·8	1·3	1·0	·74	·731	·640
,,		× 3/8	·56	·47	·40	·35	·31	·28	·25	·23	…	8·1	5·7	3·9	2·8	2·1	1·6	1·2	·88	…	…
3	× 3	× 5/16	·68	·56	·48	·42	·37	·34	·31	·28	·26	7·4	5·1	3·4	2·5	1·8	1·3	1·0	·74	·902	·612
,,		× 3/8	·80	·66	·57	·50	·44	·40	·36	·33	·31	8·7	6·0	4·1	2·9	2·2	1·6	1·2	·88	…	…
,,		× 7/16	·92	·76	·66	·58	·51	·46	·42	·38	·35	10	6·9	4·7	3·4	2·5	1·8	1·4	1·0	…	…
3	× 4	× 3/8	1·4	1·2	·99	·87	·77	·69	·63	·58	·53	12	6·6	4·3	3·1	2·3	1·6	1·2	·92	1·24	·571
,,		× ½	1·8	1·5	1·3	1·1	1·0	·91	·82	·75	·70	16	8·6	5·6	4·1	3·0	2·1	1·5	1·2	…	…
3½	× 3½	× 3/8	1·1	·92	·79	·69	·61	·55	·50	·46	·42	11	8·5	6·0	4·3	3·3	2·6	2·0	1·5	1·05	·717
,,		× 7/16	1·3	1·1	·91	·79	·71	·64	·58	·53	·49	13	9·8	6·9	5·0	3·8	3·0	2·3	1·8	…	…
,,		× ½	1·4	1·2	1·0	·90	·80	·72	·65	·60	·56	14	11	7·9	5·6	4·3	3·4	2·6	2·0	…	…
4	× 3	× 3/8	·83	·69	·59	·52	·46	·42	·38	·35	·32	12	10	7·9	5·8	4·4	3·5	2·9	2·3	·863	·875
,,		× ½	1·1	·90	·78	·68	·60	·54	·49	·45	·42	15	13	10	7·6	5·7	4·6	3·8	3·0	…	…
4	× 4	× 3/8	1·4	1·2	1·0	·90	·80	·72	·66	·60	·56	13	11	8·1	6·1	4·6	3·7	2·9	2·3	1·21	·814
,,		× ½	1·9	1·6	1·3	1·2	1·1	·95	·86	·79	·73	17	14	11	7·9	6·0	4·9	3·8	3·0	…	…
4	× 5	× 3/8	2·2	1·8	1·6	1·4	1·2	1·1	1·0	·92	·85	15	12	8·6	6·2	4·7	3·8	3·0	2·3	1·55	·762
,,		× ½	2·9	2·4	2·1	1·8	1·6	1·4	1·3	1·2	1·1	19	16	11	8·1	6·1	4·9	3·9	3·0	…	…
5	× 3	× 3/8	·85	·71	·61	·53	·47	·43	·39	·36	·33	14	13	11	9·2	7·5	6·0	4·9	4·1	·828	1·14
,,		× ½	1·1	·93	·80	·70	·62	·56	·51	·47	·43	19	17	15	12	9·8	7·8	6·4	5·4	…	…
5	× 3½	× ½	1·5	1·3	1·1	·94	·84	·75	·69	·63	·58	20	18	16	13	10	8·2	6·8	5·6	·990	1·12
5	× 4	× ½	2·0	1·6	1·4	1·2	1·1	·98	·89	·82	·75	21	19	16	13	10	8·3	6·8	5·7	1·16	1·08
6	× 3	× 3/8	·87	·73	·62	·54	·48	·44	·40	·36	·33	17	16	14	13	11	9·1	7·6	6·4	·795	1·40
,,		× ½	1·1	·95	·81	·71	·63	·57	·52	·48	·44	22	20	19	17	14	12	10	8·4	…	…
6	× 4	× ½	2·0	1·7	1·4	1·2	1·1	1·0	·91	·83	·77	24	23	21	18	15	13	11	8·7	1·13	1·34
7	× 3½	× ½	1·6	1·3	1·1	·98	·87	·78	·71	·65	·60	26	25	23	21	19	17	14	13	·924	1·65

NOTES TO THE ABOVE TABLE.

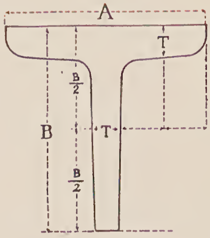
(1) The tabulated "Distributed Loads" are calculated for a working stress of 7½ tons per square inch by the following formula:—

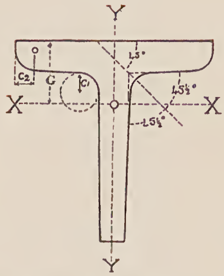
Safe Load (tons) = 8 × Section Modulus (xx) × Working Stress ÷ Span (inches).

(2) The "Safe Loads as Struts" are calculated on the same basis as the Safe Loads on Broad Flange Beams, Table A, page 157 ("Fidler's," Factor of Safety 5, "Fixed Ends").

T
Misc. Shapes.
Misc. Tables.
Prices etc.
Code.
Grey Mill.
Tests etc.
R.I.B.A.
Photos.

BRITISH STANDARD TEES.—Continued.





FURTHER NOTES ON BRITISH STANDARD TEES.

(1) It is important to give the dimensions of Tees in the correct order, namely Table (A) × Stalk (B) × Thickness (T).

(2) In addition to the British Standard Sections of Tees, the following are still in common use and freely stocked :—

$5'' \times 5'' \times \frac{1}{2}''$	Reference No. 1091.	Code Word :	"Lawyer."
$5'' \times 5'' \times \frac{5}{8}''$	,, ,, 1092.	,, ,,	"Leader."
$6'' \times 6'' \times \frac{1}{2}''$	,, ,, 1093.	,, ,,	"Leakage."
$6'' \times 6'' \times \frac{3}{4}''$	,, ,, 1094.	,, ,,	"Ledger."

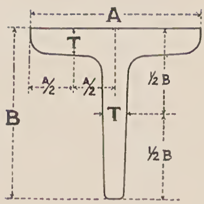
(3) Unlike most other sections, Tees cannot be varied in thickness by "lifting" or "squeezing" the rolls. Many sections of the same height and width as the standard sections but of greater thickness are still readily obtainable.

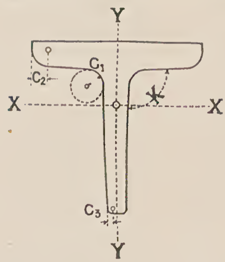
(4) Code words for sections of thicknesses not given in the table can be made up by adding the supplementary code words given in § 9, page 221 ("Code"), as explained in the notes to the tables of Angles.

(5) Tees are obtainable of smaller sizes than the standard sections, down to about $\frac{1}{2}'' \times \frac{1}{2}'' \times \frac{1}{8}''$.

METRIC STANDARD TEES.

(Table on opposite page.)





NOTES ON METRIC STANDARD TEES.

TAPER.

Those sections of which the table (A) is twice the height of the stalk (B) have 2 % taper on the flanges and 2 % taper on each side of the stalk. The remaining sections, viz. those of which the table and stalk are equal, have 2 % taper on the flanges and 4 % on each side of the stalk. In the former case the angle K is 92·4° and in the latter 93·5°.

RADII

$C_1 = T$ (thickness). $C_2 = \frac{1}{2}$ T. $C_3 = \frac{1}{4}$ T.

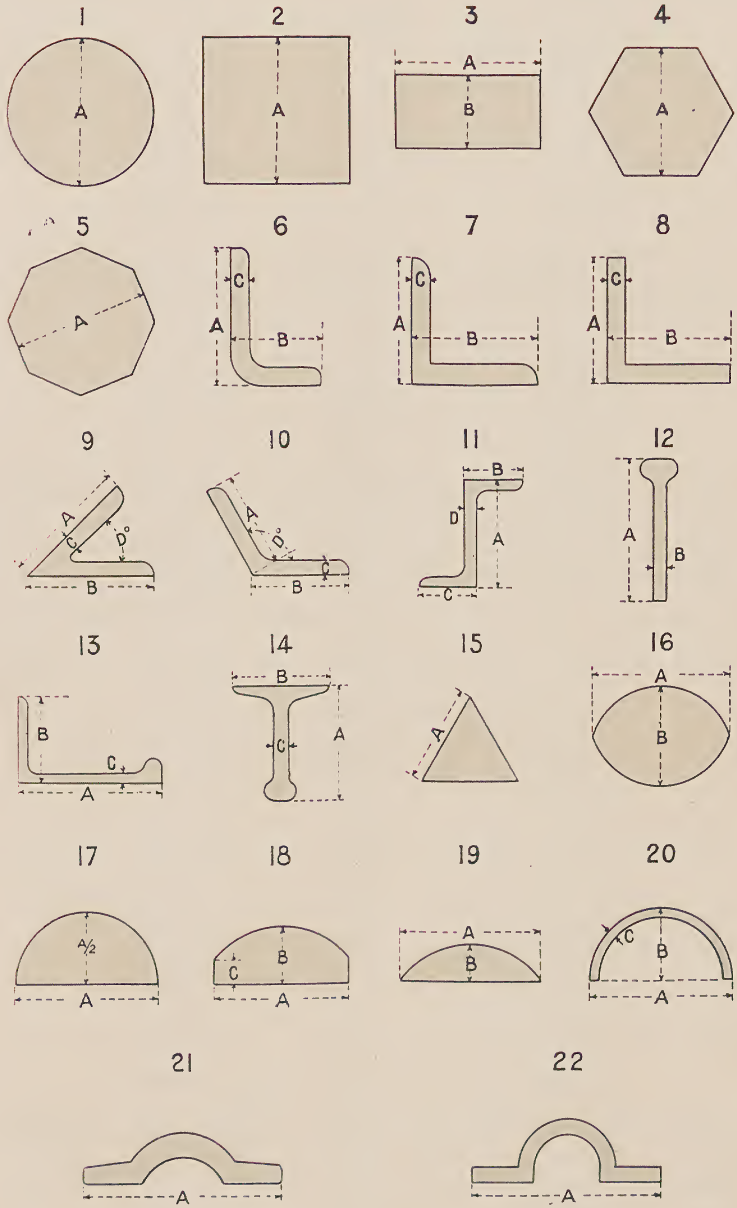
METRIC STANDARD TEES.

English Dimensions: Size A × B	Thickness	Weight per foot	Sectional Area	Reference No. and Code Word		Metric Dimensions: Size A × B	Thickness	Weight per metre	Sectional Area
Inches.	Ins.	Lbs.	Sq. Ins.			Mm.	Mm.	Kilos.	Cm²
·79 × ·79	·118	0·58	·174	1102	Labyrinth	20 × 20	3	0·87	1·12
·98 × ·98	·138	0·86	·254	1106	Lackey	25 × 25	3·5	1·28	1·64
1·18 × 1·18	·157	1·18	·350	1110	Ladder	30 × 30	4	1·76	2·26
1·38 × 1·38	·177	1·56	·460	1114	Ladle	35 × 35	4·5	2·32	2·97
1·57 × 1·57	·197	1·98	·584	1118	Lager	40 × 40	5	2·94	3·77
1·77 × 1·77	·217	2·45	·724	1122	Laird	45 × 45	5·5	3·64	4·67
1·97 × 1·97	·236	2·97	·877	1126	Laity	50 × 50	6	4·42	5·66
2·36 × 1·18	·217	2·43	·719	1130	Lambkin	60 × 30	5·5	3·62	4·64
2·36 × 2·36	·276	4·16	1·23	1134	Lament	60 × 60	7	6·19	7·94
2·76 × 1·38	·236	3·11	·921	1138	Lampoon	70 × 35	6	4·63	5·94
2·76 × 2·76	·315	5·56	1·64	1142	Lamprey	70 × 70	8	8·27	10·6
3·15 × 1·57	·276	4·15	1·23	1146	Lance	80 × 40	7	6·17	7·91
3·15 × 3·15	·354	7·12	2·11	1150	Landau	80 × 80	9	10·6	13·6
3·54 × 1·77	·315	5·33	1·58	1154	Landscape	90 × 45	8	7·93	10·2
3·54 × 3·54	·394	8·94	2·65	1158	Landslip	90 × 90	10	13·3	17·1
3·94 × 1·97	·335	6·30	1·86	1162	Landsman	100 × 50	8·5	9·38	12·0
3·94 × 3·94	·433	11·0	3·24	1166	Language	100 × 100	11	16·3	20·9
4·72 × 2·36	·394	8·87	2·63	1170	Lantern	120 × 60	10	13·2	17·0
4·72 × 4·72	·512	15·5	4·59	1174	Larboard	120 × 120	13	23·1	29·6
5·51 × 2·76	·453	12·0	3·53	1178	Larceny	140 × 70	11·5	17·8	22·8
5·51 × 5·51	·591	20·9	6·18	1182	Larder	140 × 140	15	31·1	39·9
6·30 × 3·15	·512	15·5	4·57	1186	Lasso	160 × 80	13	23·0	29·5
7·09 × 3·54	·571	19·4	5·74	1190	Lather	180 × 90	14·5	28·8	37·0
7·87 × 3·94	·630	23·8	7·04	1194	Lattice	200 × 100	16	35·4	45·4

Misc. Shapes.
Misc. Tables.
Prices etc.
Code.
Grey Mill.
Tests etc.
R.I.B.A
Photo

MISCELLANEOUS ROLLED STEEL SHAPES.

[See Description on opposite page.]



[Continued overleaf, page 212.]

MISCELLANEOUS ROLLED STEEL SHAPES.

[See illustrations on opposite page and on page 212 overleaf.]

The illustrations on pages 210 and 212 give an idea of the diversity of shapes rolled in steel. Mistakes often arise through the use of ambiguous descriptions or through stating dimensions in other than the conventional order.

If shapes are ordered by the following names or code words and the dimensions given in the same order as on the illustrations, such mistakes can hardly occur. Some indications as to the uses of the various shapes, range of existing sizes etc., are given below, but for more detailed information, readers should apply for special lists or catalogues.

Unless otherwise stated, the code words will be understood to indicate that sections are to be of mild rolled steel. The following supplementary code words can be added where iron bars are required:—

"MACARONI" =	Puddled	Wrought-Iron.	
"MADMAN" =	,,	,,	Staffordshire Crown.
"MADONNA" =	,,	,,	Best Staffordshire.
"MAGAZINE" =	,,	,,	B.B. ,,
"MAGGOT" =	,,	,,	B.B.B. ,,
"MAGICIAN" =	,,	,,	Best Yorkshire.

The practical limits of sizes rolled in wrought-iron are:—Rounds up to $3\frac{1}{2}''$ diameter; Squares up to $3\frac{1}{2}''$ sides; Flats up to $8''$ wide; Angles up to 8 united inches; Tees up to $4'' \times 4''$; Channels up to $4''$ deep; Plates up to $4'$ wide.

1. "ROUND(S)." (Code Word: "MAGNATE.") From $\frac{1}{8}''$ to $12''$ in diameter. Larger sizes are *forged*. Small lots of sizes up to about 8 inches round can usually be obtained from stock, in steel.

2. "SQUARE(S)." (Code Word: "MAGNETISM.") From $\frac{1}{8}''$ to $6''$ square. Larger sizes are *forged*. Small lots of sizes from about $\frac{1}{4}''$ to $4''$ square can usually be obtained from stock, in steel.

3. "FLAT(S)." (Code Word: "MAIDEN.") From $\frac{1}{2}'' \times \frac{1}{8}''$ to $24'' \times \frac{5}{8}''$. Flats over $18''$ wide are not at present readily obtainable in this country. Small lots of flats, from $\frac{1}{2}''$ to $6''$ wide in iron and from $\frac{1}{2}''$ to $12''$ wide in steel, are usually obtainable from stock, in ordinary thicknesses. Plates with rolled edges, known as "Universal Plates," are made on the Continent up to 40 inches wide.

4. "HEXAGON." (Code Word: "MAINLAND.") From about $\frac{3}{8}''$ to $4''$. A few small sizes, under $2''$, are stocked in limited quantities.

5. "OCTAGON." (Code Word: "MAINMAST.") From about $\frac{3}{8}''$ to $2''$.

6. "ROUND-BACKED ANGLE."* (Code Word: "MAJESTY.") Various sizes are made, from about $1'' \times 1'' \times \frac{1}{8}''$ to $4'' \times 4''$ etc. Also a few unequal-sided sections as $6'' \times 2\frac{1}{2}''$, $7'' \times 3''$ etc.

7. "SQUARE-ROOT ANGLE."* (Code Word: "MALADY.") From $\frac{1}{2}'' \times \frac{1}{2}'' \times \frac{1}{8}''$ to about $5'' \times 5'' \times \frac{1}{2}''$.

8 "SQUARE-EDGE AND SQUARE-ROOT ANGLE."* (Code Word: "MALLOW.") From $\frac{1}{2}'' \times \frac{1}{2}'' \times \frac{1}{8}''$ to about $5'' \times 5'' \times \frac{1}{2}''$.

9. "ACUTE ANGLE."* (Code Word: "MALARIA.")

10. "OBTUSE ANGLE."* (Code Word: "MALICE.") A few sizes are made, to various angles (D) and of various sizes up to $5'' \times 5''$. Two sizes in *fairly* common use are $2'' \times 2'' \times \frac{1}{2}'' \times 135^\circ$ and $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}'' \times 96^\circ$.

11. "ZED BAR" OR "Z BAR." (Code Word: "MALTSTER.") There are 8 British Standard Sections from $3''$ to $10''$ deep (A) and 11 Metric Standard Sections from 30 mm. to 200 mm. deep.

12. "BULB PLATE" OR "BULB FLAT." (Code Word: "MANDATE.") There are 7 British Standard Sizes from $6''$ to $12''$ deep (A) and 15 Metric Standard Sizes from 130 to 400 mm. deep.

13. "BULB ANGLE." (Code Word: "MANGLE.") There are 20 British Standard Sizes from $4'' \times 2\frac{1}{2}''$ to $12'' \times 4''$ and 6 Metric Standard Sizes from 130×65 mm. to 200×85 mm.

14. "BULB TEE." (Code Word: "MANIFESTO.") There are 6 British Standard Sizes from $7''$ to $12''$ deep and 13 Metric Standard Sizes from 150 to 400 mm. deep.

15. "TRIANGLE." (Code Word: "MANTEL.") From $\frac{1}{4}''$ to $2\frac{3}{8}''$.

16. "OVAL." (Code Word: "MANSE.") From $\frac{1}{2}'' \times \frac{5}{16}''$ to about $3'' \times 1''$. One or two small sizes from $\frac{5}{8}'' \times \frac{5}{16}''$ to $1'' \times \frac{1}{2}''$ are stocked, in limited quantities.

17. "HALF ROUND." (Code Word: "MAPLE.") From $\frac{1}{2}''$ to $4''$ wide.

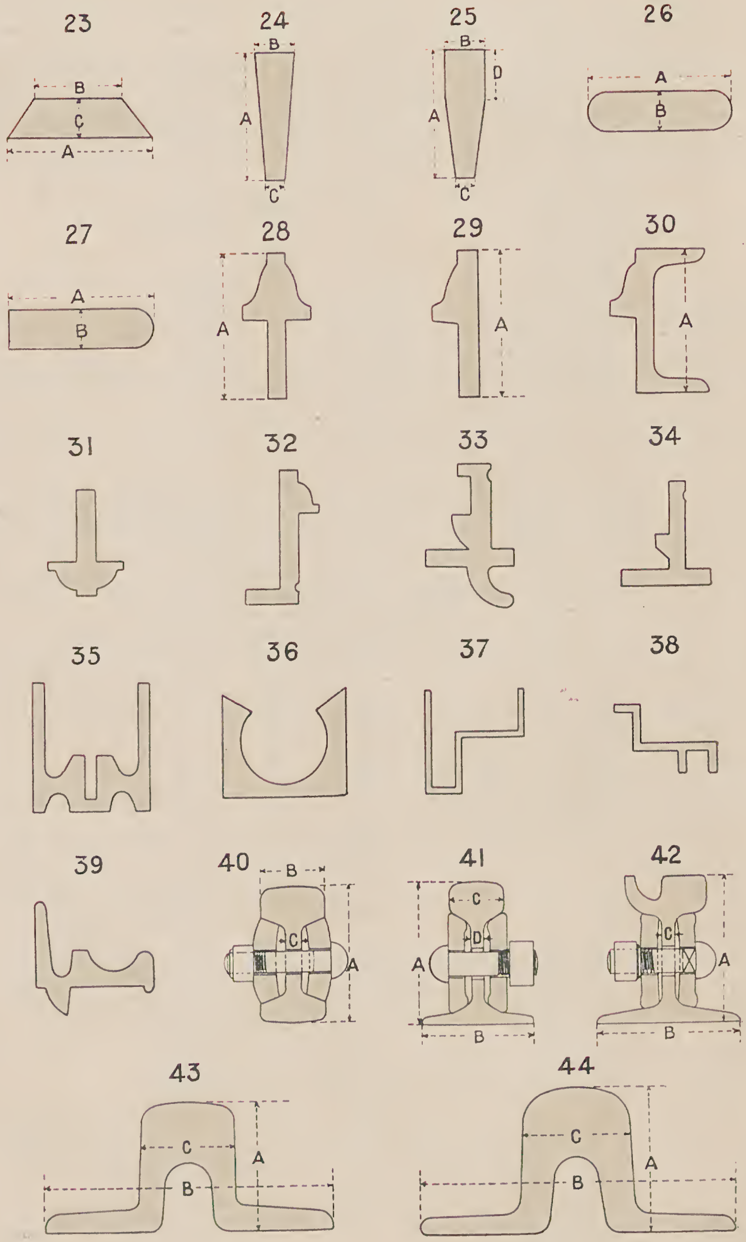
18. "SQUARE-EDGE CONVEX." (Code Word: "MARBLE.") From $\frac{1}{2}''$ to $9''$ wide (A). Sizes from $1''$ to $3''$ wide are rolled in a great variety of thicknesses (B, C). Shapes similar to this, but thicker on one side than the other, are also made and usually called "Convex Tyre Bars." (Code Word: "MARGARINE.") Sizes range from about $1\frac{3}{4}'' \times \frac{3}{8}'' \times \frac{1}{2}'' \times \frac{1}{4}''$ to $6'' \times \frac{1}{2}'' \times \frac{5}{8}'' \times \frac{3}{8}''$. A few sizes are stocked.

* These five types of angles (Nos. 6 to 10) are "special" sections. That is to say, very few sizes are made by more than one maker. Prompt delivery of small lots cannot be relied upon. Unless otherwise stated, the two flanges of angles of all types are of the same thickness.

[Continued overleaf, page 213.]

MISCELLANEOUS ROLLED STEEL SHAPES.—Continued.

[See Description on opposite page.]



MISCELLANEOUS ROLLED STEEL SHAPES.—Continued.

[See illustrations on opposite page and on page 210 overleaf.]

19. "FEATHER-EDGE CONVEX." (Code Word: "MARINER.") From ½" to 6" wide, and various thicknesses (B).

20. "HOLLOW HALF-ROUND." (Code Word: "MARKSMAN.") 2" × 1" × ⅜", 2½" × 1¼" × ½", 3" × 1½" × $\frac{9}{16}$" etc. These sections are to be distinguished from the crescent-shaped sections called "Thimble" bars (Code Word: "MARMALADE") which are made from ½" to 2" wide.

21, 22. "FUNNEL RINGS." (Code Word: "MARQUIS.") Various sizes. Orders or inquiries for these sections must be accompanied by a drawing.

23.* "BEVEL-EDGED BAR." (Code Word: "MARRIAGE.") 1½" × 1¼" × ½" and similar sizes. A few sizes are made bevelled on one side only. (Code Word: "MARROW.")

24.* "'FIRE' OR 'SCREEN' BAR," with equally tapered sides. (Code Word: "MARSH.") Various sizes from about 1¼" × ½" × ¼" to 4½" × 1" × ¾". A few sizes are made tapered on one side only. (Code Word: "MARSUPIAL.")

25. "'FIRE' OR 'SCREEN' BAR," with square shoulders. (Code Word: "MARTELLO.") 4" × 1" × ⅜" × 1¼", 2½" × ⅝" × ¼" × 1" etc.

26. "TWO ROUND-EDGE FLAT." From ⅝" × ¼" to about 2" × 1". Common sizes range from 1" × $\frac{3}{16}$" to 1¾" × ¾". These are made in Puddled Wrought-Iron (Code Word: "MARTEN") and in Steel of Welding Quality for Tyres (Code Word: "MARTINET"). The width on flat is about ⅛" to ⅜" less than the extreme width (A) according to size etc.

27. "ONE ROUND-EDGE FLAT." Common sizes range from 1" × $\frac{7}{16}$" to 4" × ¾". These are made in Puddled Wrought-Iron (Code Word: "MARTINMAS") and in Steel of Welding Quality for Tyres (Code Word: "MARTYR"). The width on flat is about ⅛" to $\frac{3}{16}$" less than the extreme width (A) according to size etc. A few sizes from 1½" to 2⅝" wide are stocked in limited quantities.

28. "DOUBLE-MOULDED SASH BAR." (Code Word: "MARVEL.") Standard sizes are 1¼", 1½" and 1¾" deep (A).

29. "SINGLE-MOULDED SASH BAR." (Code Word: "MASSACRE.") Standard sizes are 1¼", 1½" and 1¾" deep (A).

30. "DOUBLE-FLANGED SASH BAR." (Code Word: "MASTIFF.") Usual size, 1½" deep.

N.B.—Illustrations 28, 29 and 30 (Sash Bars) are one-half actual size. These sections are usually ordered by depth only, but makers' patterns differ somewhat in weight and thickness.

31 to 39. These are examples of irregular shapes, drawn one-half actual size. They are mostly special casement sections.

40. "'BULL-HEADED' RAIL." (Code Word: "MATADOR.") The British Standard Sections weigh 60, 65, 70, 75, 80, 85, 90, 95 and 100 lbs. per yard respectively.

"MATCHLOCK" denotes: "B.S. Bull-Headed Rail(s) weighing .... lbs. per yard."

41. "'FLAT-BOTTOMED' OR 'FLANGE' RAIL." (Code Word: "MATINEE.") There is a large range of available sizes from about 7 to 100 lbs. per yard. There are 17 British Standard Sections ranging in weight from 20 to 100 lbs. per yard.

"MATRON" denotes: "B.S. Flange Rail(s) weighing .... lbs. per yard."

42. "GROOVED-HEAD TRAMWAY RAIL." (Code Word: "MATTOCK.") There are 5 British Standard Sections, namely 6½" × 6½" × 90 lbs., 6½" × 7" × 95 lbs., 6½" × 7" × 100 lbs., 7" × 7" × 105 lbs. and 7" × 7" × 110 lbs. per yard respectively, with 5 corresponding sections, 6 lbs. per yard heavier in each instance, for use on curves.

"MATURITY" = "British Standard Tramway Rail(s) weighing .... lbs. per yard."
"MAUSOLEUM" = "B.S. Tram-Rail(s) for use on curves weighing .... lbs. per yard."

Tramway Rails similar in type to Fig. 41, but of greater depth, are also used. They are known as "Solid-Head Girder Tramway Rails." (Code Word: "MAXIM.")

43, 44. "BRIDGE RAILS." (Code Word: "MAYDAY.") Sizes weighing about 14, 16, 18 and 20 lbs. per yard are stocked. A readily available size as used for Crane runways measures 2⅝" × 6" × 2" × 56 lbs. per yard. (Code Word: "MEDALLIST.") Another fairly common size is 3" × 6½" × 2¼" × 70 lbs. per yard. (Code Word: "MEDIATOR.") These two sizes are represented in Figs. 43 and 44 to a scale of 3 inches = 1 foot. Small lots are generally obtainable from stock in lengths up to 40 feet.

N.B.—Rails are only quoted for to a given drawing or template and specification. But these can of course be dispensed with if British Standard Sections and Specification are specified. The stated weights of rails do not include fishplates etc.

The following supplementary code words may be useful:—

"MEDICINE" = "(To conform to) British Standard Specification of Tests and Conditions."
"MEMORIAL" = "Including fishplates, but not bolts and nuts or spikes."
"MENACE" = "Including fishplates, bolts and nuts (and ....)."
"MENIAL" = "United States Standard Flange Rail weighing .... lbs. per yard."
"MERCER" = "(To conform to) United States Standard Specification of Tests etc."
"MERINO" = "Prussian State Flange Rail weighing .... kilos per metre."
"MERLIN" = "(To conform to) Prussian State Railway Specification of Tests etc."

* Some makers call Sections 23 and 24 "Bevel Bars," without discrimination.

Misc. Tables.
Prices etc.
Code.
Grey Mill.
Tests etc.
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Photos

SHEARING AND BEARING VALUES OF RIVETS (OR BOLTS) IN TONS PER RIVET.

TABLE A. Taking safe shearing strength as 4 tons per square inch.

Diameter of Rivet.	Sectional Area.	Least Thickness of Plate which will give full Shearing Value of Rivet. Single Shear.	Double Shear.	Safe Single Shear per Rivet.	Bearing Value per Rivet on Thin Plates. $\frac{1}{4}''$ ·25″	$\frac{5}{16}''$ ·31″	$\frac{3}{8}''$ ·38″	Safe Double Shear per Rivet.	Bearing Value per Rivet on Thin Plates. $\frac{5}{16}''$ ·31″	$\frac{3}{8}''$ ·38″	$\frac{7}{16}''$ ·44″	$\frac{1}{2}''$ ·5″	$\frac{9}{16}''$ ·56″
Inches.	Sq. Ins.	Inches.	Inches.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.
$\frac{3}{8}$	·110	·147	·221	·441	...	...	...	·662	...	...	...	...	...
$\frac{1}{2}$	·196	·196	·294	·785	...	...	...	1·178	...	...	...	...	...
$\frac{5}{8}$	·307	·245	·368	1·227	...	...	...	1·841	1·56	...	...	...	...
$\frac{3}{4}$	·442	·295	·442	1·767	$1\frac{1}{2}$	...	...	2·651	1·88	$2\frac{1}{4}$	2·63	...	...
$\frac{7}{8}$	·601	·344	·515	2·405	$1\frac{3}{4}$	2·19	...	3·608	2·19	2·63	3·06	$3\frac{1}{2}$	...
1	·785	·393	·589	3·141	2	$2\frac{1}{2}$	3	4·712	$2\frac{1}{2}$	3	$3\frac{1}{2}$	4	$4\frac{1}{2}$

The value of rivets in single shear is taken as 4 tons per square inch.
The value of rivets in double shear is taken as $1\frac{1}{2}$ times the above.
The bearing value of rivets is taken as 8 tons per square inch.
Bearing values which are greater than the shearing values need not be considered and are therefore omitted for the sake of clearness.
The "least thicknesses of plates" for full shearing value are given in decimals of an inch for the equivalent thicknesses in fractions of an inch (nearest 64th), see Table B.

TABLE B. Taking safe shearing strength as 3 tons per square inch.

Diameter of Rivet.	Sectional Area.	Least Thickness of Plate which will give full Shearing Value of Rivet. Single Shear.	Double Shear.	Safe Single Shear per Rivet.	Bearing Value per Rivet on Thin Plates. $\frac{1}{4}''$ ·25″	$\frac{5}{16}''$ ·31″	$\frac{3}{8}''$ ·38″	Safe Double Shear per Rivet.	Bearing Value per Rivet on Thin Plates. $\frac{5}{16}''$ ·31″	$\frac{3}{8}''$ ·38″	$\frac{7}{16}''$ ·44″	$\frac{1}{2}''$ ·5″	$\frac{9}{16}''$ ·56″
Inches.	Sq. Ins.	Inches.	Inches.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.
$\frac{3}{8}$	·110	$\frac{9}{64}$	$\frac{7}{32}$	·331	...	...	...	·497	...	...	...	...	...
$\frac{1}{2}$	·196	$\frac{13}{64}$	$\frac{19}{64}$	·589	...	...	...	·883	...	...	...	...	...
$\frac{5}{8}$	·307	$\frac{1}{4}$	$\frac{3}{8}$	·920	...	...	...	1·381	1·17	...	...	...	...
$\frac{3}{4}$	·442	$\frac{19}{64}$	$\frac{7}{16}$	1·325	1·13	...	...	1·988	1·41	1·69	1·97	...	...
$\frac{7}{8}$	·601	$\frac{11}{32}$	$\frac{33}{64}$	1·804	1·31	1·64	...	2·706	1·64	1·97	2·3	2·63	...
1	·785	$\frac{25}{64}$	$\frac{13}{32}$	2·356	$1\frac{1}{2}$	1·88	$2\frac{1}{4}$	3·534	1·88	$2\frac{1}{4}$	2·63	3	3·38

The value of rivets in single shear is taken as 3 tons per square inch.
The value of rivets in double shear is taken as $1\frac{1}{2}$ times the above.
The bearing value of rivets is taken as 6 tons per square inch.
Bearing values which are greater than the shearing values need not be considered and are therefore omitted for the sake of clearness.
The "least thicknesses of plates" for full shearing value are given to the nearest 64th of an inch; for the equivalent thicknesses in decimals of an inch, see Table A.

METRIC EQUIVALENTS Etc.

[Explanation of Table on pages 216 and 217 overleaf.]

METRIC EQUIVALENTS.

With the exception of the "Approximate Prices," the whole of the metric equivalents in the following tables have been specially calculated from the following fundamental data:—

1 metre = 39·370113 inches.
1 kilogram = 2·2046223 pounds.
1 gallon = 4·5459631 litres.

These equivalents were laid down by an Order in Council dated 19th May, 1898, in pursuance of the Weights and Measures (Metric System) Act, 1897, and represent the latest* official determinations of the Standards deposited at the Board of Trade.

EQUIVALENT OF GALLON.

The equivalents of the Gallon in cubic inches etc. are based on the assumption that:

1 gallon = 277·420 cubic inches.

The Gallon is the only Imperial Standard Measure of Capacity. It contains 10 Pounds Avoirdupois of distilled water, weighed in the air against brass weights, at the temperature of 62° Fahrenheit, the Barometer being at 30 inches.

There is no statutory equivalent of the Gallon in cubic measure except for the sale of Gas, for which purpose the legal equivalent of the Gallon is 277·274 cubic inches (Sale of Gas Act, 1859). In August, 1890, the Board of Trade found the equivalent of the Gallon to be 277·463† cubic inches, but this equivalent is not legalised. The Comité International des Poids et Mesures (Paris) reported last year (1905) that 1 cubic centimetre of distilled water at the temperature of 4° Centigrade, Barometer at 760 mm., weighs 0·999974 gramme.

Hence, the equivalent of the Gallon would be 277·420 cubic inches approximately.

This being the latest and most authoritative determination has been adopted in compiling the tables overleaf.

APPROXIMATE PRICES.

The stated equivalents of English prices in francs and marks are based on the assumptions that £1 = 25·10 to 25·20 francs
and £1 = 20·40 to 20·50 marks, respectively.

UNITED STATES.

The legal equivalents in the United States are:—

1 metre = 39·37 inches.
1 kilogram = 2·2046 pounds.

For practical purposes, these standards are substantially identical with those of the English Board of Trade. But the Standard Measures of *Capacity* differ materially:—

The United States Gallon for Liquid Measure contains 231 cubic inches and, for Dry Measure, 268¾ cubic inches.

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* At time of publication of this handbook (July, 1906).

† One cubic inch of distilled water (pressure, temperature etc. as above) was found to weigh 252·286 grains.

METRIC EQUIVALENTS Etc.

To convert	Into	Multiply by	Log	Reciprocal	Log
LENGTH.					
64ths of an inch ...	Millimetres	0·39688	598653	2·51969	401347
32nds ,, ,, ...	,, ...	0·79375	899683	1·25984	100317
16ths ,, ,, ...	,, ...	1·58750	200713	0·62992	799287
8ths ,, ,, ...	,, ...	3·17500	501743	0·31496	498257
Inches	Millimetres	25·4000	404833	0·03937	595167
,, ...	Centimetres	2·54000	,,	0·39370	,,
,, ...	Metres	0·02540	,,	39·3701	,,
Feet	Metres	0·30480	484015	3·28084	515986
Yards	,, ...	0·91440	961136	1·09361	038864
Links	Inches	7·92	898725	0·12626	101275
,, ...	Metres	0·20117	303558	4·97098	696442
Chains	Feet	66	819544	0·01515	180456
,, ...	Metres	20·1168	303558	0·04971	696442
Miles	Kilometres	1·60934	206649	0·62137	793352
AREA.					
Square inches ...	Square millimetres ...	645·159	809667	0·00155	190333
,, ,, ...	Square centimetres ...	6·45159	,,	0·15500	,,
Square feet ..	Square metres ...	0·09290	968029	10·7639	031971
Square yards ...	,, ,, ...	0·83613	922272	1·19599	077728
Acres	Square metres ...	4046·85	607117	0·00025	392883
,, ...	Hectares	0·40468	,,	2·47106	,,
,, ...	Square yards	4840	684845	0·00021	315155
Square miles ...	Acres	640	806180	0·00156	193820
,, ,, ...	Square kilometres ...	2·58998	413297	0·38610	586703
Square kilometres ...	Hectares	100	...	...	...
Hectares	Square metres ...	10,000	...	...	...
VOLUME.					
Cubic inches ...	Cubic millimetres ...	16387·0	214500	0·00006	785500
,, ,, ...	,, centimetres ...	16·3870	,,	0·06102	,,
Cubic feet	Cubic metres	0·02832	452044	35·3148	547956
Cubic yards	,, ,, ...	0·76455	883408	1·30795	116592
Pints	Litres	0·56825	754537	1·75980	245463
Gallons	Cubic inches	277·420	443138	0·00360	556862
,, ...	Cubic feet	0·16054	205594	6·22882	794406
,, ...	Litres	4·54596	657626	0·21998	342374
Bushels	Gallons	8	903090	0·125	096910
,, ...	Cubic feet	1·28435	108684	0·77860	891316
,, ...	Litres	36·3677	560716	0·02750	439284
Hectolitres	Litres	100	...	...	...
Cubic metres ...	,, ...	(1000)	...	...	...
WEIGHT.					
Ounces	Grammes	28·3495	452546	0·03527	547454
Pounds	Kilogrammes	0·45359	656666	2·20462	343334
Cwts.	,, ...	50·8023	705884	0·01968	294116
Tons	,, ...	1016·05	006914	0·00098	993086
Quintal	,, ...	100	...	...	...

METRIC EQUIVALENTS Etc.

To convert	Into	Multiply by	Log	Reciprocal	Log
WEIGHT PER FOOT Etc.					
Pounds per foot ...	Kilos. per metre ...	1·48817	172651	0·67197	827349
Pounds per yard ...	,, ,, ,, ...	0·49606	695530	2·01590	304470
PRESSURE, STRESS Etc.					
Pounds per sq. inch	Kilos. per square mm.	0·00070	846999	1422·33	153001
,, ,, ,,	,, ,, ,, cm.	0·07031	,,	14·2233	,,
,, ,, ,,	,, ,, ,, metre	703·071	,,	0·00142	,,
Pounds per sq. foot	Kilos. per square mm.	...	688637	204,816	311363
,, ,, ,,	,, ,, ,, cm.	0·00049	,,	2048·16	,,
,, ,, ,,	,, ,, ,, metre	4·88244	,,	0·20482	,,
Tons per sq. inch ...	Kilos. per square mm.	1·57488	197247	0·63497	802753
Tons per sq. foot ...	Lbs. per square inch	15·5556	191886	0·06429	808115
,, ,, ,, ...	Kilos. per square cm.	1·09367	038885	0·91436	961115
,, ,, ,, ...	,, ,, ,, metre	10936·7	,,	0·00009	,,
Tons per acre ...	Kilos. per hectare ...	2510·71	399797	0·00040	600203
Bushels per acre ...	Hectolitres per hectare	0·89867	953599	1·11276	046401
Foothead of water ...	Lbs. per square inch ...	0·43256	636044	2·31183	363957
VELOCITY.					
Miles per hour ...	Feet per second ...	1·46667	166331	0·68182	833669
Kilometres per hour	,, ,, ,, ...	0·91135	959683	1·09728	040317
APPROXIMATE PRICES.					
Shillings per ton ...	Francs per 1,000 kilos.	1·2352	091730	0·80960	908270
		to			
		1·2401	093457	0·80639	906543
Shillings per ton ...	Marks per 1,000 kilos.	1·0039	001686	0·99612	998314
		to			
		1·0088	003810	0·99127	996190
Pounds (st.) per ton	Pence per lb.	0·10714	029963	$9\frac{1}{3}$	970037
WORK.					
Foot-pounds ...	Kilogram-metres ...	0·13825	140680	7·23302	859320
Inch-tons	,, ,, ...	25·8076	411747	0·03875	588253
Foot-tons	,, ,, ...	309·691	490928	0·00323	509072
MOMENTS OF INERTIA Etc.					
$Inches^4$	$Centimetres^4$	41·6230	619333	0·02403	380667
$Inches^3$	$Centimetres^3$	16·3870	214500	0·06102	785500

EXPLANATION.

To convert English Measures into Metric Measures, multiply by the figures in the column headed "Multiply by." To reverse the process, *i.e.* to convert Metric Measures into English Measures, multiply by the figures in the column headed "Reciprocal."

E.g. 1 inch =25·4000 millimetres.
1 millimetre= 0·03937 inch.

For Notes on the Metric System and mode of calculation, see page 215.

SCHEDULE FOR FORMING ROUGH ESTIMATES OF COST OF IRON AND STEEL WORK.

Prices of iron and steel fluctuate and the cost of engineering work varies still more. Consequently, the following approximate prices, although generally on the safe side, can only be regarded as very rough estimates. Except in the case of relatively trifling orders, they may be assumed to cover cost of carriage.

Basis price of Broad Flange Beams	£7 to £8 per ton.
,, ,, British Standard Joists	Ditto
,, ,, German Standard Joists	£6 10s. to £7 10s. per ton.

N.B.—The term "basis price" denotes the price for sizes etc. which do not carry an extra. In the case of rolled steel beams, sections from 4 to 12 inches deep, excepting such sections as 9″ × 7″, 10″ × 8″ etc., are usually supplied at basis price. The above prices are for delivery direct from mills; the approximate cost of rolled steel beams when delivered from local stock will be about £1 10s. per ton extra.

Steel Stanchions, with base etc. (plain H sections)...	£10 to £12 per ton.
,, ,, ,, ,, ,, (built-sections) ...	£11 to £13 ,,
Compound Girders, up to 16″ deep	£9 to £10 ,,
,, ,, over 16″ deep	£9 10s. to £11 ,,
Plate and Angle Girders	£10 to £12 ,,
Plain Cast-iron Columns	£6 10s. to £8 ,,
Rolled Steel Angles, Channels etc. in ordinary lengths and sizes	£6 10s. to £7 ,,
Rolled Steel Tees, Rounds, Flats and Squares etc. in ordinary lengths and sizes	£7 to £7 10s. ,,

WORKMANSHIP Etc.

Angle Cleats, Tee Stiffeners etc. in small lots and short lengths	12s. to 15s. per cwt.
Fishplates, without holes	Ditto
Holes in angles, plates etc.	2s. to 3s. per dozen.
,, in rolled steel beams	3s. to 4s. ,,
Rivets (closed)	10s. to £1 per 100.
Loose Rivets (steel)	10s. to 15s. per cwt.
Common Bolts and Nuts for ordinary connections ...	4d. each.
,, ,, ,, ...	16s. to £1 per cwt.
Plain Notches to rolled steel beams	5s. to 15s. per notch.
Bevel Cuts ,, ,, ,, ...	5s. to 10s. per cut.
Cutting to "exact" lengths (when not done at mills)	Ditto
Hoisting and fixing of girders and stanchions ...	£1 5s. to £3 per ton.
Galvanized Corrugated Sheets (when lapped) ...	2d. to 3d. per sq. foot.
Steel Rails (small lots for cranes etc.)	£7 to £9 per ton.
Best Wrought-iron Bars (Staffordshire marked bars)	£8 to £10 ,,
Best Yorkshire Iron ("Farnley" brand)	22s. to 25s. per cwt.
Steel Plates	£8 to £10 per ton.
Steel or Iron Sheets ($\frac{1}{8}$″ thick and under)	£9 to £11 ,,

STEELWORK, PREPARED READY FOR ERECTION.

Girders and Stanchions for warehouse	£10 to £12 per ton.
Steelwork for foundry, with crane girders and roof principals	3s. 6d. to 5s. per sq. foot.
Steel Roofwork with columns for large sheds ...	1s. to 2s. per sq. foot.
Large, single-storied iron buildings	Ditto
Plain Roof Principals, for 20 feet span	£3 to £3 10s. each.
,, ,, ,, for 50 ,, ,, ...	£10 to £15 ,,

STANDARD LIST OF EXTRAS FOR BROAD FLANGE BEAMS.

[SUBJECT TO ALTERATION WITHOUT NOTICE.]

CUTTING TO LENGTHS.

Unless otherwise instructed, Broad Flange Beams are cut to specified lengths within a margin of 1½ inches over, free of charge. They can be cut, at works, to within ⅛ inch "under or over" at 5s. per ton extra, if required. If the specified length must not be exceeded, or *vice versa*, the margin must be increased to ¼ inch, viz. "¼ inch under" or "¼ inch over" respectively.

COLD STRAIGHTENING.

Broad Flange Beams are always straightened (when necessary) before leaving the works, free of charge.

ROLLING MARGIN.

4 % on sections up to 16 inches and 5 % on larger sections. The margin to be taken under or over.

QUALITY OF STEEL. See page 231 ("Tests").

MINIMUM AND MAXIMUM LENGTHS.

All Broad Flange Beams can be supplied in any lengths up to about 56 feet. The smaller sections can be rolled in greater lengths, if required. Lengths under 4 feet or over 40 feet are charged extra as below, in addition to any extra cost of freight or carriage that may be involved on long and heavy pieces.

SECTION EXTRAS.

Sections from 7″ to 12″ deep	sold at basis price.
Sections over 12″ to 16″ ,, ...	10s. per ton extra.
,, ,, 16″ to 22″ ,, ...	20s. ,, ,,
Section 24″ × 12″	25s. ,, ,,
,, 26″ × 12″	30s. ,, ,,
,, 30″ × 12″	40s. ,, ,,

LENGTHS EXCEEDING 40 FEET.

Sections up to 15″ deep	1s. 0d. per ton, per ft., extra.
Sections over 15″ ,, ...	1s. 6d. ,, ,, ,,

LENGTHS UNDER 4 FEET DOWN TO 18 INCHES.

4s. per ton extra, or more if required cut to "exact" lengths.

CUTTING TO "EXACT" LENGTHS.

Within ⅜″ under or over	2s. 6d. per ton.
,, ⅛″ ,, ,, ...	5s. 0d. ,,

DRILLING.

Round holes in Web ...	2d. each	Oval holes ...	3d. each.
,, ,, in Flanges ...	4d. ,,	,, ,, ...	6d. ,,

Subject to a minimum charge of 2s. per ton.

PAINTING.

Ordinary iron oxide	2s. 6d. per ton, per coat.
Linseed oil or tar	Ditto
Red lead	5s. 0d. per ton, per coat.

Prices etc.
Code.
Grey Mill.
Tests etc.
R.I.B.A.
Photos

TELEGRAPHIC CODE WORDS.

(1) The code words given in this book may be used in conjunction with any of the following public codes. Code words are assigned to each of the latter, in order that the sender may indicate, when necessary, which of them he is using*:—

A.B.C. (Fourth Edition)... "MERMAID." A.B.C. (Fifth Edition)... "MIDGET."
Lieber's Code... "MILDEW." Lieber's Numeral Code... "MILITARY."
Western Union ... "MILLER." Moreing and Neal's ... "MINION."
British Engineering Standards Coded Lists ... "MINORITY."
Stevens' Engineering Telegraph Code (Second Edition) ... "MODESTY."

(2) The code words given in this book do not require to be supplemented by such descriptions as "*Channel section disguise*" etc. The word "*disguise*" denotes: "...... *Rolled Steel Channel(s) of British Standard Section* $10'' \times 3\frac{1}{2}''$ *etc.*," and cannot be interpreted to mean any other shape or size.

(3) Buyers who have correspondents abroad and desire to use the code words in this book to supplement their existing code, should add a special word to their ordinary code to signify "*Some of the code words in this telegram are to be interpreted from H. J. Skelton & Co.'s handbook.*"

It is suggested that the word "SKELCODE" be used for this purpose.

(4) Pains have been taken to avoid giving any code words in this book which might possibly cause confusion when used in conjunction with the various public codes enumerated above. But some business-houses may find code words in this book which also occur and have another signification in their ordinary private codes. When this is found to be the case, any risk of misinterpretation could be avoided by inserting one or other of the following code words, according to circumstances:—

WATERMAN or WAVES ..	"The following code word is to be interpreted in accordance with H. J. Skelton & Co.'s Code."
WAVELET or WAVERER ..	"The following code word is *not* to be interpreted in accordance with H. J. Skelton & Co.'s Code."

(5) The responsibility for incorrect execution of cabled orders or for other possible loss or inconvenience resulting from ambiguity or mutilation of telegrams in transmission, rests with the purchaser.

(6) The code words used in this book for the various standard rolled steel shapes are as follows. Unless otherwise stated, they all imply that sections of mild rolled steel are required:—

"ABBESS"	to "ALKALI"	Page 118.	Broad Flange Beams.
"BACHELOR"	to "BAZAAR"	„ 186.	Joists (British Standard Sections).
"CACTUS"	to "CEREMONY"	„ 191.	„ (Metric Standard Sections).
"DAFFODIL"	to "DOMICILE"	„ 192.	Channels (British Standard Sections).
"EAGLE"	to "EMBRYO"	„ 194.	„ (Metric Standard Sections).
"FABIAN"	to "FRIVOLITY"	„ 196.	Equal-sided Angles (British Standard Sections).
"GABLE"	to "GUARDIAN"	„ 198.	„ „ (Metric Standard Sections).
"HABIT"	to "INDIGNITY"	„ 200.	Unequal-sided Angles (British Standard Sections).
"INEBRIATE"	to "INSANITY"	„ 185.	Plated Beams.
"JACKET"	to "JUGGLER"	„ 204.	Unequal-sided Angles (Metric Standard Sections).
"KAISER"	to "KYANISER"	„ 206.	Tees (British Standard Sections).
"LABYRINTH"	to "LATTICE"	„ 209.	„ (Metric Standard Sections).
"LAWYER"	to "LEDGER"	„ 208.	„ (Special Sections).
"MACARONI"	to "MERLIN"	„ 211.	Miscellaneous Shapes.
"MERMAID"	to "MODESTY"	See § 1 above.	

* The first word in the telegram should be "SKELCODE," indicating the use of the code words in this handbook (see § 3), and the second word should indicate what public code is used in conjunction therewith.

(7) The following additional words will be found of occasional utility :—

SKELCODE .. See § 3 on opposite page.

WADDING .. Telegraph to-day's basis price for Broad Flange Beams.

WADER .. To-day's basis price for Broad Flange Beams is ....shillings per ton.

WAGER .. Broad Flange Beams.

WAGGERY .. Put the following into stock at works pending further instructions which follow by mail.

WAGGONER.. (Further) workmanship will be required before despatch; particulars follow by letter.

WAGTAIL .. Have opened necessary credit with the following London bankers........... with whom please communicate.

WAIST .. Material to be painted with (one coat of) red oxide paint.

WAITER .. ,, ,, ,, ,, linseed oil.

WALKER .. ,, ,, and oiled (one coat of each).

WALLET .. Two coats.

WALNUT .. To be cut to specified lengths within ⅛th of an inch under or over.

WALTZ .. ,, ,, ,, ¼th of an inch.

WANDERER . ,, ,, ,, ⅜ths of an inch under or over.

WARBLER .. ,, ,, ,, 1½ inches over, nothing under.

WARDEN .. Margin to be taken under.

WARDROBE.. ,, ,, over.

WARNER .. Steel to have a minimum tenacity of 26 tons per square inch, with 20 % minimum elongation in 8 inches.

WASPS .. Must be of British manufacture if possible.

WATCHER .. All dimensions to be in accordance with H. J. Skelton & Co.'s handbook.

WATCHMAN . Purchase from H. J. Skelton & Co.

WATERMAN, WAVES, WAVELET, WAVERER. See § 4 on opposite page.

(8) *The following code words refer to the drawings of standard connections published in this book.* It is not suggested that it will often be necessary or practicable to give orders for structural work by cable, the most useful method of saving time in such cases being to cable an order for the plain beams required (*vide* code word WAGGERY), and to forward drawings etc. showing the workmanship by mail.

WAYLAYER .. To be fitted with standard web cleats.*

WEALD .. ,, ,, ,, and lower flange cleat.*

WEAPON .. ,, ,, ,, and upper flange cleat.*

WEASEL .. ,, ,, ,, and flange cleats.*

WEATHER .. ....pair(s) of standard fishplates, with holes and bolts, to suit B.F. Beam Section............

WEAVER .. To be drilled for standard fishplates.*

WEDDING .. At one end only.

WEDLOCK .. At both ends.

WEEPER .. To be fitted with standard stanchion base(s).

WEEVIL . ,, ,, ,, ,, and cap(s), light pattern.

WELCOME .. ,, ,, ,, ,, ,, heavy pattern.

WELDER .. No web cleats required in stanchion cap(s).

WELFARE .. No cover plate required in stanchion cap(s).

WELKIN .. Do not provide any holes for connection bolts (except in.. .....).

WELLSPRING ,, ,, ,, ,, (in........).

WESTWARD.. Drill holes for connection bolts prior to despatch where practicable; but, where in doubt, omit holes and bolts.

WHALER .. (With) flange plates riveted top and bottom; total thickness of plates ....inches, width....inches.

WHEEDLER . Rivets to be flush countersunk for a distance of....inches from each end.

(9) The use of the following code words is explained in the notes to the table of British Standard Angles, pages 196 etc.

WHEEZE ..	3/16" thick (however).	WHIGS ..	¼" thick (however).
WHIRLIGIG .	⅜" ,, ,,	WHIRLPOOL .	½" ,, ,,
WHISKY ..	⅝" ,, ,,	WHISPER ..	¾" ,, ,,
WHIST ..	⅞" ,, ,,	WHITEWASH .	1" ,, ,,
WHITING ..	Thickness (however) to be......		

* In using the code words marked with an asterisk, it is obviously necessary to add whether "ataone end only or "at both ends."

Code.
Grey Mill.
Tests etc.
R.I.B.A.
Photos

MANUFACTURE OF ROLLED STEEL BEAMS.

The ordinary rolling mill as it has existed from the middle of the eighteenth century to the present day, consists essentially of two parallel cylindrical rolls driven in opposite directions, so as to draw the piece of metal through and gradually reduce it to the required shape.

Fig. 1 shows a rolling mill for converting "ingots" into "blooms," known as a "blooming" or "cogging" mill. It will be observed that there are six

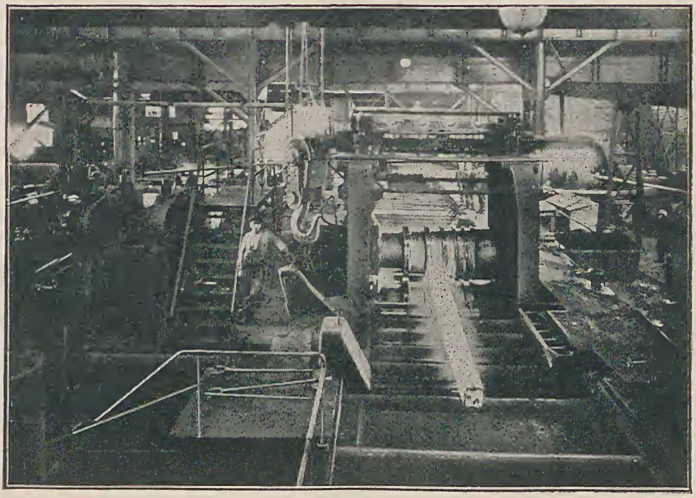


Fig. 1.

grooves of diminishing size. The "bloom" passes backwards and forwards through the various grooves until it has been reduced to the required "section."

Fig. 2 shows the "finishing pass" in the rolls as required for producing an H section. The arrows show how the flanges are formed. It will be observed that the only direct pressure exerted by the rolls is on the web of the beams; the flanges are formed by squeezing the metal into grooves in the rolls.

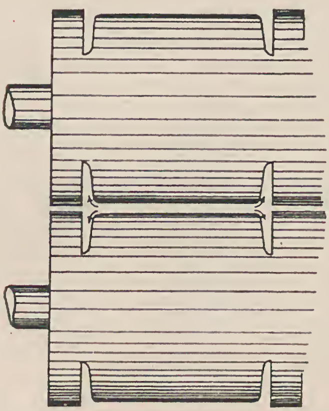


Fig. 2.

It is this feature in the process of manufacture which determines the maximum practicable height and flange width of sections when produced in an ordinary mill. The deeper the grooves in the rolls, the greater the difficulty in inducing the metal to "flow" into and completely fill the grooves. All experienced buyers know how often the flanges of such sections as $8'' \times 6''$, $9'' \times 7''$ etc. are $\frac{1}{4}''$ to $\frac{1}{2}''$ under the stipulated width. A little consideration will also make it clear why manufacturers insist on the inner surfaces of the flanges being tapered and rounded off.

If, for example, the junction between web and flange were not suitably rounded off, it is easy to see that the metal would refuse to "flow" into the grooves. Either the engines would be brought to a standstill or the rolls break; in fact, these are not infrequent occurrences in rolling heavy and difficult sections in an ordinary mill. The "section" must also be so designed as to enter the rolls without excessive shock to the mill, and to leave them without any tendency to stick. The latter occurrence is serious and is most likely to occur in rolling sections with wide and thin flanges such as tramway rails etc.

Another manufacturing feature which may be mentioned is that H sections having thick flanges and thin webs, have an unpleasant habit of curling up on leaving the rolls etc. This does not happen in the case of beams rolled in a "Grey" mill: they rarely require subsequent straightening.

The facts mentioned impose unavoidable limitations on the size and shape of sections intended to be rolled in an ordinary mill. Some improvements have nevertheless been introduced.

For example, the rolling of a 16″ joist was deemed until recently a remarkable achievement, though now there are several works producing 24″ joists with flanges 7 and $7\frac{1}{2}$ inches wide. Improvements have been made in shape also. Sharper patterns of steel rails are rolled nowadays with thinner webs and reduced curves; manufacturers have ceased to demand that the inner surfaces of rolled steel "angles" should be tapered. Similarly in regard to rolled steel joists.* The features of improvement are shown in the annexed diagram (Fig. 3). The inclination of the flanges and radii of the curves are diminished, the web reduced and the flanges increased in width and thickness.

All these changes have been made with two objects:—

(1) To increase and flatten the surfaces available for riveting etc.

(2) To distribute the metal at a greater average distance from the centre of the section. In this way the efficiency, *i.e.* carrying power of the section, whether used as a beam or as a strut, is increased without increasing its weight.

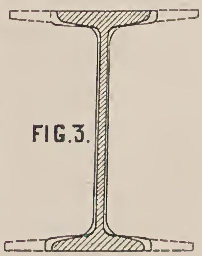


FIG. 3.

However, the improvements which have been made in the design of rolled steel shapes during the last thirty years are trifling as compared with the remarkable advance which was rendered possible by the introduction of the "Grey" mill. This mill was invented by an Englishman, Mr. Henry Grey, who emigrated to the United States and eventually became manager of the

* It is instructive to compare the recently standardised British sections with the corresponding sections in the United States and Germany. The latter having been standardised at an earlier date when manufacturers were less skilled, are naturally less scientific in design than the British sections. For example, the inclination of the flanges of United States joists is $16\frac{2}{3}$ %, while that of the B.S. joists is only 14 %. The German sections all have relatively narrower and thinner flanges with thicker webs than the British sections. Neither the United States nor Germany have any wide-flanged sections like 8″ × 6″, 9″ × 7″ etc.

An interesting feature in the evolution of rolled steel beams is the temporary retrogression in design which followed the supplanting both of cast-iron by wrought-iron and subsequently of wrought-iron by steel. There are still in use old cast-iron girders, precisely similar in shape to Broad Flange Beams, except that the metal is thicker and the flanges not tapered. Similarly, between 1860 and 1880, solid wrought-iron girders were regularly made 20″ × 10″ and occasionally 30″ × 10″ at the Butterley Works in Derbyshire. They were made by welding up T pieces 10″ × 10″ with a flat bar in between.

Grey Mill.
Tests etc.
R.I.B.A.
Photos.

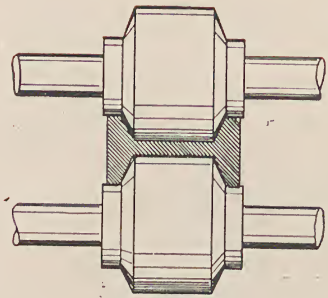


FIG. 4.

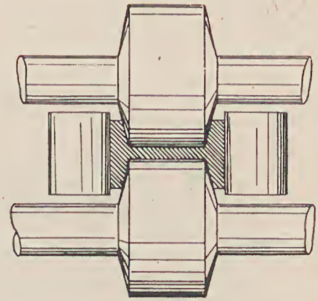


FIG. 6.

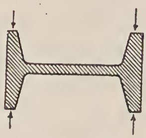


FIG. 5.

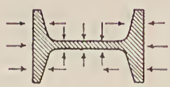


FIG. 7.

Homestead Works of the well-known Carnegie Steel Company. The essential features of this mill are illustrated in the diagrams Figs. 4 to 7. There are two "stands" of rolls placed as close as possible. One of these stands consists of the pair of rolls shown in Fig. 4. The other stand consists of two vertical and two horizontal rolls (Fig. 6). The "Grey" mill is served by a special "blooming" mill which reduces the ingots to a suitable size and shape. The "blooms," thus formed, pass backwards and forwards through the "Grey" mill, the various rolls being brought closer at each pass, until the blooms have been reduced to the required shapes. The pair of rolls shown in Fig. 4 are so arranged that, in the last or "finishing" pass, only the edges of the flanges are rolled as shown by the arrows in Fig. 5. The functions of the rolls in Fig. 6 are obvious from the diagram. The pressure exerted by these rolls on the various parts of the section is shown in Fig. 7.

The advantages of the "Grey" mill, leaving aside questions of economical working,* may be summarised as follows:—

(1) The difficulty encountered in inducing metal to fill deep grooves in ordinary rolls is practically eliminated by the "Grey" mill, as the sizes of Broad Flange Beams show.

(2) The practical features of the ordinary rolling mill which influence the *shape* of sections, irrespective of size, are greatly diminished by the "Grey" mill. Broad Flange Beams have only 9% taper on the flanges as compared with the 14 % taper of ordinary joists. The edges of the flanges are rolled square instead of being rounded off. By suitable modifications of the mill, rolled steel joists can be produced without any taper on the flanges, with practically square corners throughout and with thinner webs than it is possible to produce in an ordinary mill.

---

* It may be mentioned, however, that the initial cost of turning rolls for the "Grey" mill is very much less than that of the rolls required to produce a similar shape in an ordinary mill.

(3) The finish of beams produced by the "Grey" mill is greatly superior to that obtained from the ordinary mill.

(*a*) On referring to Fig. 2, it will be easy to see why ribs or "fins" are often formed down the centre of the flanges of ordinary joists. In the "Grey" mill, however, the vertical rolls in Fig. 6 produce a clean, flat surface which is an important advantage where the ends of beams rest on stanchions or where connections of any kind have to be made. (*b*) Users of Broad Flange Beams have often commented on their relative freedom from "scale" which saves considerable trouble and expense in preparing the beams for painting. This is due to the flanges being rolled in the manner described and to jets of water being directed on the rolls and the beam throughout the rolling operation.* (*c*) The edges of the flanges, being rolled as shown in Fig. 5, are clean and sharply defined.

(4) Perhaps the most important advantage of the "Grey" mill is the fact that the section is rolled all over. Pressure is exerted on both sides of the flanges, between the vertical rolls and the horizontal rolls (Fig. 6), and also on the edges of the flanges, so that the metal is thoroughly sound in every part of the section.

It has been pointed out above that, in rolling H sections and similar shapes in an ordinary mill, the only direct pressure exerted by the rolls is on the web of the beams; the flanges are formed by squeezing the metal into grooves in the rolls. When rolling round and square shapes in an ordinary mill, these are turned over at every pass so that the section is rolled on all sides and the plastic metal welded into a solid homogeneous bar. This is obviously impracticable in rolling an H section. As a consequence, if test pieces are cut from the edges of the flanges, the metal invariably proves to be of inferior quality to that of the web, both in ductility and in tenacity. In rolling comparatively easy† sections, this undesirable feature is not so strongly marked. But when the flanges are wide and the grooves in the rolls correspondingly deep, it is only with difficulty that the metal is induced to fill the grooves at all. Very little pressure is exerted either on the sides or edges of the flanges and, whereas the metal in the web is solid and compact, the metal in the flanges, especially towards the edges, remains comparatively "spongy." For this reason, it would appear that the practicable limits of the ordinary rolling mill have now been reached, if not already exceeded; and that, as consumers continue to demand larger and wider-flanged sections, the universal adoption of the "Grey" mill is only a matter of time. No doubt, enterprising manufacturers might, in course of time, produce all the existing sections of Broad Flange Beams in an ordinary rolling mill, but the difficulty of producing flanges of uniform width is very great and the difficulty of producing such flanges in sound metal is still greater. This applies quite as much to difficult sections of square shape and thin metal like the smaller sizes of Broad Flange Beams as to large and heavy beams.

It is hardly likely that manufacturers will attempt to produce in an ordinary mill any Broad Flange Beam or similar section in competition with

* In an ordinary mill, the shape passes from one groove to another and from one set of rolls to another. But, in the "Grey" mill, there is only one groove as it were. Hence a tendency to overheat the rolls if not cooled in the manner described.

† It is not possible without lengthy explanation to describe exactly what constitutes an "easy" section. Heavy and wide-flanged sections are obviously difficult sections. But it is not merely a question of absolute weight, size or width. A standard 9″ × 7″ joist is an easier section to roll than Broad Flange Beam 7″ × 7″; an 8″ × 6″ joist is more difficult than a 12″ × 6″ joist.

" Grey " mills, as the percentage of waste and various other expenses and risks would be excessive. It would nevertheless be prudent for readers who have occasion to specify Broad Flange Beams to stipulate that they must be rolled in a " Grey " mill.

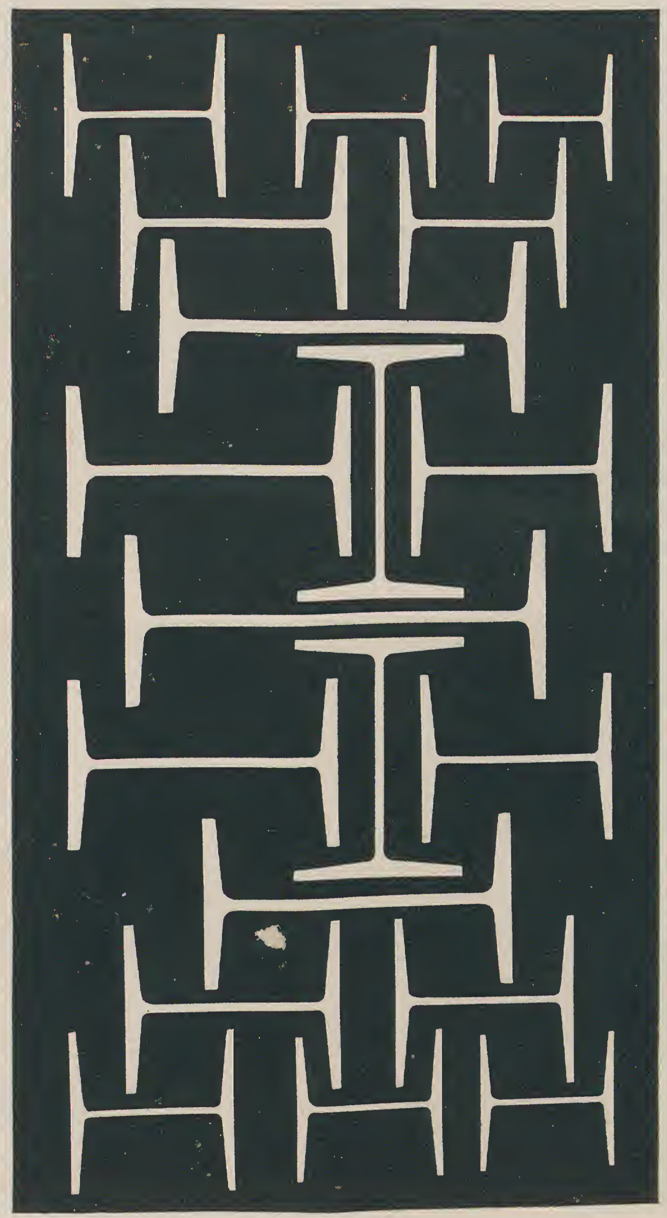
Similarly, although it would be rash to assert that the science of roll-turning has reached its limit in this direction, buyers should at present be chary of attempting to persuade manufacturers to increase the dimensions of the present standard sizes produced in ordinary rolling mills. There would be no great advantage in obtaining a $20'' \times 8''$ joist in lieu of $20'' \times 7\frac{1}{2}''$ or $8'' \times 8''$ in lieu of $8'' \times 6''$ if the condition of the extra metal were such as to be practically valueless.

Efforts should rather be directed towards stimulating all manufacturers to put down " Grey " mills—or mills of a still better type if they can devise such. Manufacturers are naturally slow to adopt new processes until there is a strong and unmistakable demand, and the surest way to attain the desired end in this case is to specify Broad Flange Beams freely, not necessarily stipulating that they shall be supplied by any particular manufacturer, but merely that they shall be rolled in the only existing type of mill which is capable of producing Broad Flange Beams properly, viz. in a " Grey " mill.

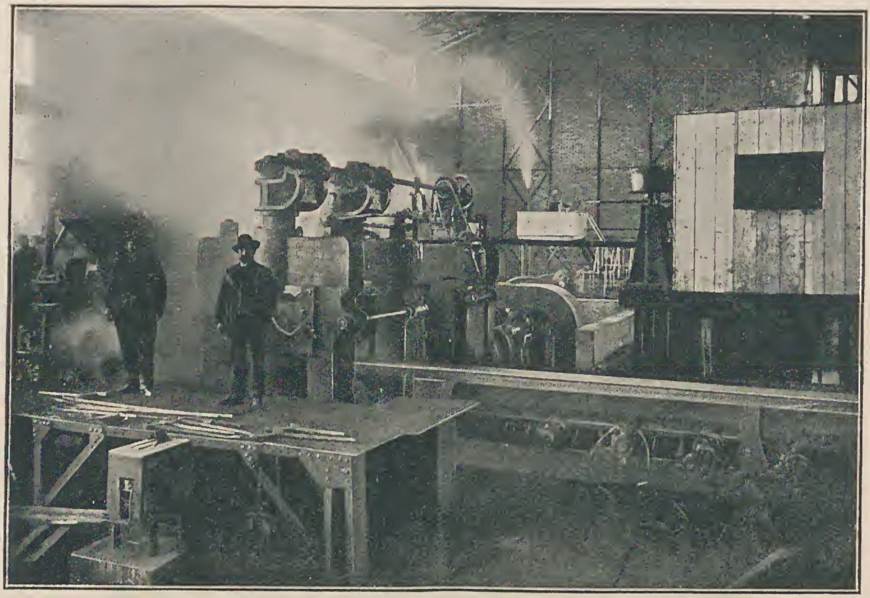
The " Grey " mill has one feature troublesome to manufacturers but which is rather an advantage from the consumer's point of view. The early attempts to produce " Grey " beams were discouraging on account of the large percentage of " wasters " (*i.e.* faulty bars which had to be scrapped).

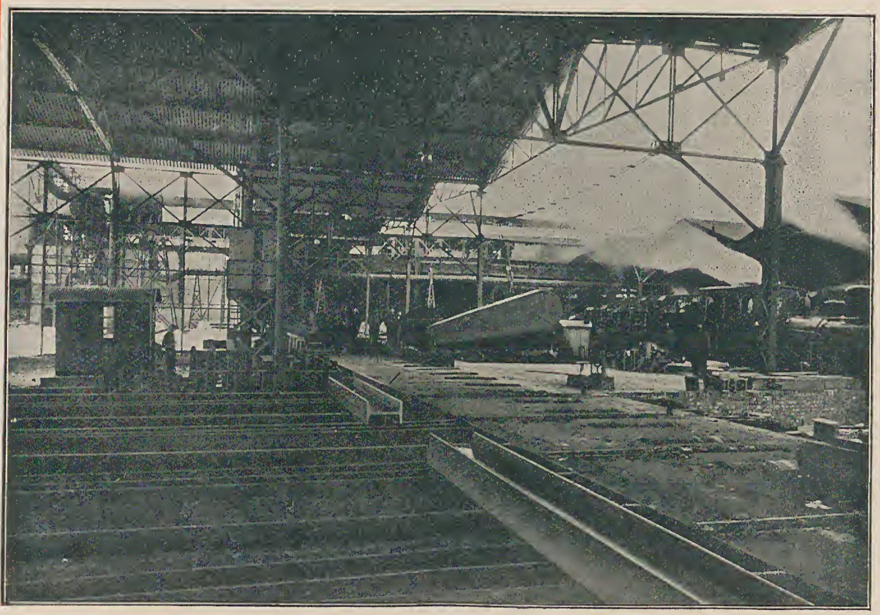
It was soon discovered that ingots, slightly defective or irregular in chemical or mechanical composition but which could be successfully rolled down in an ordinary horizontal rolling mill, were automatically rejected by the " Grey " mill in this way. This difficulty still exists, but by the exercise of extreme care in the steel-making department to produce steel of first-class, uniform quality, the percentage of waste need be little or no greater than that in an ordinary mill.

PHOTOGRAPH OF CUTTINGS OF BROAD FLANGE BEAMS, SHOWING CLEAN FINISH.

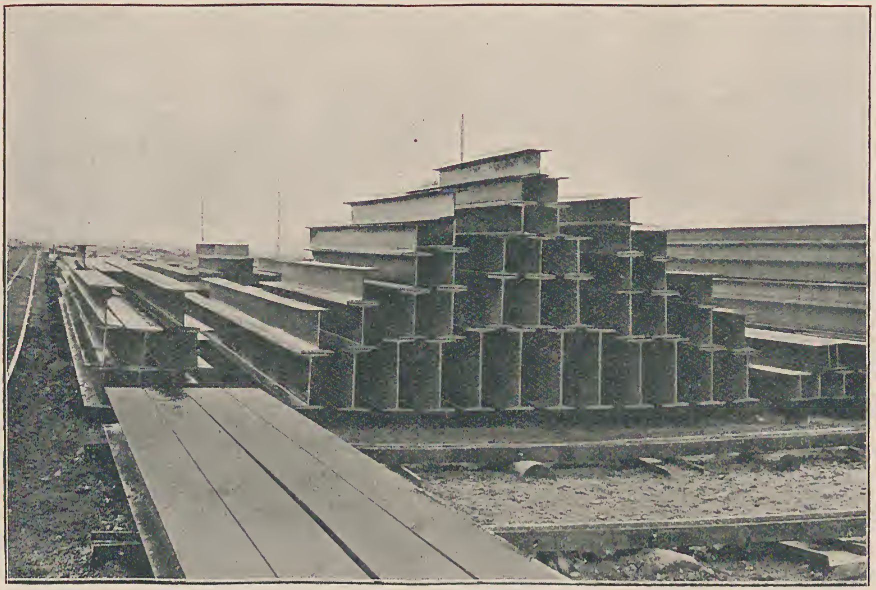


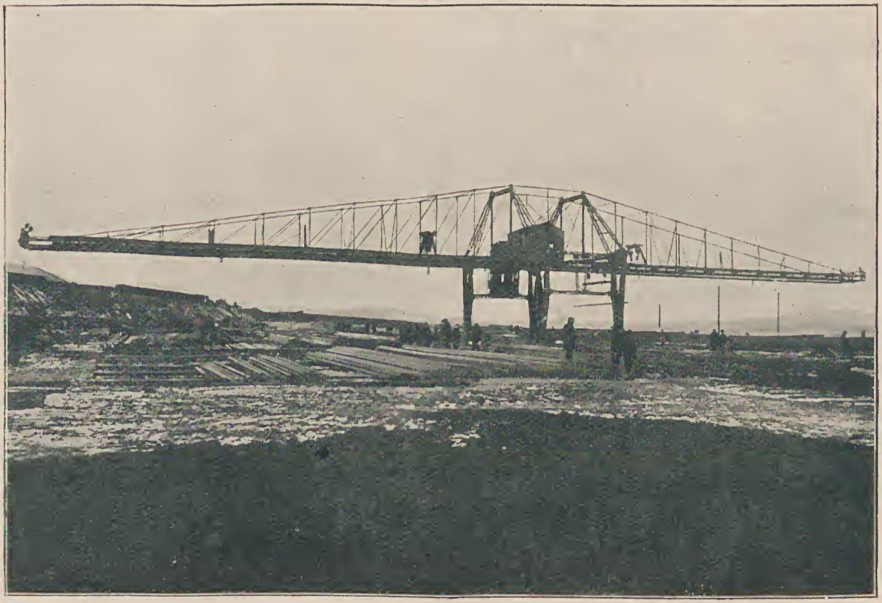
BROAD FLANGE BEAMS IN COURSE OF MANUFACTURE.





BROAD FLANGE BEAMS IN STOCK.





SUITABLE TESTS FOR STRUCTURAL STEEL.

Inexperienced users of rolled steel joists commonly assume that the higher the tensile strength of the steel, the better the quality. This would be correct if it were not that, by all existing methods of manufacture, increased tensile strength means extra carbon and consequent extra brittleness.

Phosphorus and some other impurities have the same effect as carbon, *i.e.* they increase the hardness, tenacity and brittleness.*

There is also a prevalent belief that steel of high tensile strength produces a stiffer girder than another of lower tensile strength, *i.e.* that the latter beam will give a greater deflection. This also is an entire fallacy; the deflection of a steel beam will be precisely the same for all grades of steel in practical use.†

That grade of steel is the "best" which is most suitable for the purpose for which it is intended. In the case of steel intended for use in boiler-making, or which is to be subjected to heat treatment and after manipulation of various kinds, the softer the steel the better. Rails, tyres etc. require to have a hard wearing surface so that a relatively hard steel is used. As regards steel for use in bridges and structural work generally, opinions differ as to what is the most suitable quality, but there has been an unmistakable tendency of late years in favour of lowering the carbon percentage and tensile strength, especially among engineers well acquainted with the practical conditions of steel manufacture. Thus the compression-members in a famous steel bridge were stipulated to be of steel having a minimum tensile strength of 34 tons per square inch. The same engineer's present specification of tests would render such steel liable to rejection. Many English engineers still specify 28 to 32 tons per square inch tensile strength, though some of the largest consumers in this country, including the Admiralty, allow a minimum of 26 tons per square inch. The tendency in favour of milder steel has been and will continue to be hastened by two factors:—

(1) The rapid conversion, in recent years, of leading British works to basic processes of steel-making. Formerly the open-hearth *acid* process was almost supreme in this country, and the harder qualities of steel were favoured by manufacturers as being cheaper than the softer qualities.

(2) The increasing size and weight of rolled steel sections necessitate the use of softer qualities of steel. If too hard a quality is used, it is difficult to obtain a clean well-rolled section of full dimensions.

In view of all the facts, the ordinary user of structural steel who desires to have a large area of supply, would do well to stipulate for a minimum tensile strength of about 26 tons per square inch with a minimum elongation of 20 % in 8 inches. This may be coupled with a general stipulation to the effect that mild steel of first-class quality is to be supplied. Another useful proviso, with which the best manufacturers will willingly comply, is that the buyer's inspecting engineer shall have the right of access to the works during

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* Other factors in addition to the percentage of carbon and impurities affect the tensile strength of steel. For example, an ingot from a given cast rolled into boiler plates $1\frac{1}{4}''$ thick will show a lower ultimate strength than another ingot from the same cast rolled into similar plates only $\frac{1}{2}''$ thick.

The direction of rolling, thickness of metal, finishing temperature etc. all exercise their influence on the tenacity and ductility of steel, irrespective of chemical analysis.

† Professor Unwin, in his paper on the Tensile Test of Mild Steel etc. (Proceedings of the Institute of Civil Engineers, Vol. 155, 1903-4, page 5), points out the inconsistency of stipulating a maximum tensile strength. He deals with the supposed irregularity of deflection etc. which might result from the employment in the same structure of steel members of greatly varying tensile strength as follows:—

"It may be said that a limited variation of strength is insisted on because it is undesirable in a compound structure to have plates of different strength acting together. But as the coefficient of elasticity varies very little for very great variations of tensile strength, the reason is not very convincing."

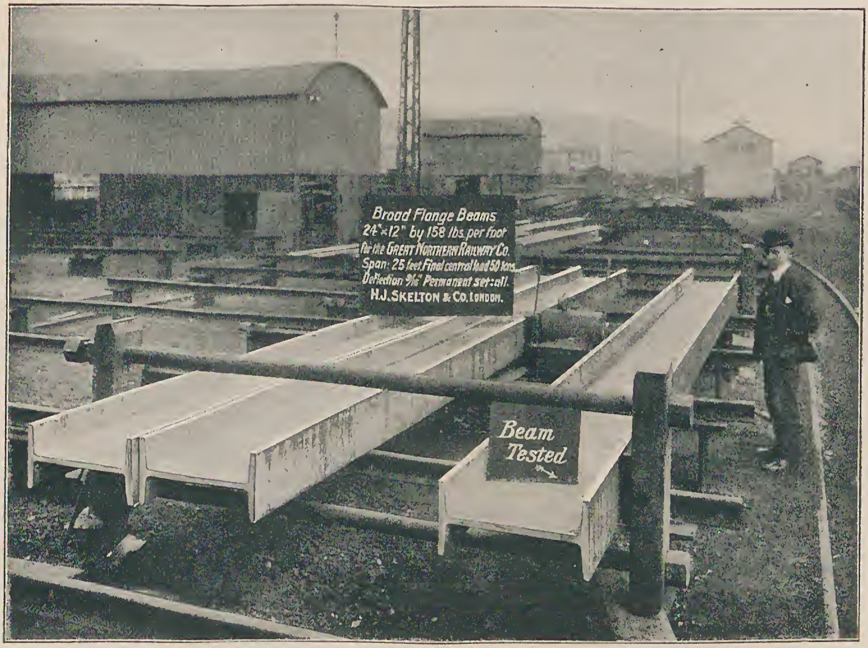
There is, in fact, no appreciable variation in the modulus of elasticity of steels ranging in tenacity from 22 to 40 tons per square inch, by whatever process the steel is made. The average value is nearly 13,000 tons per square inch.

manufacture, and shall be permitted to make a reasonable number of bending tests etc., the manufacturer to provide the necessary assistance free of charge.

In purchasing steel beams etc. from a first-class manufacturer, it is not desirable to make special stipulations as to tensile strength etc. in ordering small quantities, say lots of less than 10 to 20 tons of a size, as such orders may necessitate special rolling and consequent delay; moreover, it is hardly reasonable to expect manufacturers to incur the considerable trouble and expense of preparing test-pieces for relatively trifling orders.

Tests for Broad Flange Beams. In the case of Broad Flange Beams, considerable experience in the manufacture of various grades of steel leads to the opinion that the best and most suitable analysis of steel for structural work is one that shows under test an average tensile strength of about 26 to 27 tons per square inch, or 25 to 26 tons for the heaviest sizes, with the ductility corresponding to such tenacity in steel of the first quality.*

Buyers should therefore specify a "minimum tensile strength of 26 tons per square inch" (or preferably 25 tons per square inch for the heaviest sections) "with a minimum elongation of 20 % in 8 inches."



The above photograph shows a simple method of applying dead load tests to girders. The load is applied at the centre by means of a hydraulic testing machine which registers the load.

The beam shown in the photograph is of section 24″×12″ and was tested with a load of 50 tons at the centre of a 25 feet span, corresponding to a flange stress of 10·3 tons per square inch. The beam was not tested further as it was intended for actual use, in a public road bridge at Stevenage (Great Northern Railway Company). Similar tests have been made subsequently for the Great Northern Railway Company on beams of section 26″×12″ used in a bridge at Wood Green Station.

* Good average specimens of such steel will generally show nearly 30 % elongation in a gauge length of 8 inches as compared with the recognised standard minimum of 20 %.

LONDON BUILDING ACT AMENDMENT.

Suggestions by the Royal Institute of British Architects for the Regulation of Skeleton Buildings.

[N.B.—The following recommendations are reprinted from the *Journal of the Royal Institute of British Architects*, 6th February, 1904, by permission of the Institute.]

Notwithstanding anything contained in the principal Act requiring buildings to be enclosed with walls of the thicknesses therein defined, it shall be lawful to erect buildings of iron or steel skeleton construction subject to the following provisions:

1. The skeleton framing in any wall shall be capable of safely sustaining, independently of any brickwork, the whole weight bearing upon such wall, including the weight of such wall and the due proportion of any floors and roofs bearing thereon, together with the live load on such floors and roofs.

2. The pillars supporting all iron or steel girders that carry walls or fire-resisting floors or roofs shall be of iron or steel, and shall be completely enclosed and protected from the action of fire by a casing of brickwork or concrete or other material approved by the district surveyor. Such casing shall, on the surfaces towards the exterior of the building, be at least 8½ inches thick, and on all other surfaces at least 4 inches thick, the whole being properly bonded with the enclosing walls of the building. The term pillar shall include all columns and stanchions or an assemblage of such columns or stanchions properly riveted or bolted together.

3. The iron and steel girders (excepting in floors and staircases) shall be similarly cased with not less than 4 inches thick properly tied and bonded to the remaining work; but the flanges of the girders and the plates and angles connected therewith may approach within 2 inches of the surface of the casing.

4. Girders to support the enclosing walls shall be fixed at or within 4 feet of the floor line of each story.

5. No enclosing wall of the building shall be of less thickness than 8½ inches for the topmost 20 feet of its height, nor less than 13 inches in thickness for the remainder of its height below such topmost 20 feet, provided that window backs may, in all cases, be 8½ inches in thickness.

6. All brickwork and concrete shall be executed in cement and shall be bedded close up to the iron or steel without cavity between, and all joints shall be made full and solid. Nothing in this section shall prevent the use of stone as an external facing for buildings, provided that all work faced with stone shall be 4 inches thicker than hereinbefore provided.

7. (*a*) No steel or wrought-iron pillar shall in any part be less than ¼ inch thick, nor shall any such pillar have an unsupported length of more than 40 times its least lateral dimension, nor more than 160 times its least radius of gyration.

(*b*) The ends of all such pillars shall be faced to a true surface at right angles to the axis.

(*c*) All joints in such pillars shall be close butted with cover plates properly riveted, and, except where unavoidable, no joint shall be made except at or near the level of a girder.

(*d*) The foot of all such pillars shall have a proper baseplate riveted thereto with sufficient gusset pieces to properly distribute the load on the foundations.

(*e*) Where any such pillars are built up hollow, the cavities shall either be filled up with cement concrete or be covered in at both ends to exclude the air.

8. (*a*) In any cast-iron pillar the metal shall not be in any part of less thickness than ¾ inch nor less than one-twelfth of the least lateral dimension. Nor shall such pillar have an unsupported length of more than 20 times its least lateral dimension nor more than 80 times its least radius of gyration.

(*b*) The caps and bases of such pillars shall be in one piece with the columns, or be connected thereto with a properly turned and bored joint sufficiently fixed.

(*c*) All such pillars shall be turned or planed top and bottom to a true face at right angles to the axis.

(*d*) All joints in such pillars shall be at or near the level of a floor, and shall be fixed and made with not less than four bolts at least ¾ inch in diameter.

(*e*) The foot of all such pillars shall have such area as may be necessary to properly distribute the load on the foundations.

9. All girders that carry walls or floors or roofs shall be of wrought-iron or mild steel.

10. (*a*) All floors and all staircases (together with their enclosing walls) shall be constructed throughout of fire-resisting materials and be carried upon supports of fire-resisting materials.

(*b*) All iron and steel carrying loads used in the construction of any floor or staircase shall be protected from the action of fire by being encased to the satisfaction of the district surveyor in concrete, brickwork, terra cotta or metal, lathing and plaster or cement without any wood firrings.

11. All structural metal work shall be cleaned of all scale dust and rust and be thoroughly coated with one coat of boiled oil or paint or other approved material before erection, and after erection shall receive at least one additional coat.

12. (*a*) The dead loads of all buildings shall consist of the actual weight of walls, floors, roof, partitions, and all permanent construction.

(*b*) The live load shall consist of all loads other than dead loads.

(c) *For the purpose of calculating the loads on pillars* in buildings, the *live load* on floors shall be estimated as equivalent to the following dead load:

For dwelling houses, hotels, hospitals, lodging houses, and similar buildings, $\frac{5}{8}$ cwt. per superficial foot.

For office buildings, $\frac{7}{8}$ cwt. per superficial foot.

For places of public assembly, workshops, and retail shops, and similar buildings, 1 cwt. per superficial foot.

For buildings of the warehouse class, not less than 2 cwt. per superficial foot.

(d) The live load on the roof shall be estimated at $\frac{1}{4}$ cwt. per superficial foot measured on the surface of such roof.

13. For the purpose of determining the extreme load to be carried on pillars in buildings of more than two stories in height, a reduction of the live loads shall be allowed as follows:—

For the roof and top story the live load shall be calculated in full.

For the next succeeding lower story a reduction of 5 per cent. from the live load fixed by this section.

For the next succeeding lower story a reduction of 10 per cent.

For each succeeding lower story the amount of the reduction shall be 5 per cent. more than for the story immediately above until at the eleventh story from the top the reduction shall be 50 per cent.

For each remaining story, if any, below such eleventh story from the top the reduction shall be 50 per cent.

14. In pillars the actual working stress per square inch shall not exceed that given in the following table and in like proportion for intermediate ratios:

Where the Length divided by Least Radius of Gyration Equals	Working Stress in Tons per Square Inch of Section. Cast-Iron.	Steel.	Wrought-Iron.
160	...	2·512	2·142
140	.	2·957	2·477
120	. .	3·460	2·825
100	...	4·017	3·170
80	1·875	4·452	3·470
60	2·442	4·832	3·727
40	3·026	5·100	3·895
20	3·464	5·290	4·000

Where a pillar is built into a wall the radius of gyration of that pillar in the direction of the thickness of the wall shall be taken for the purpose of the above table.

15. The actual working stress of iron and steel (except in the case of pillars as hereinbefore set out) in tons per square inch of sectional area, shall not exceed those given in the following table:

	Tension.	Compression.	Shearing.	Bearing.
Cast-iron	$1\frac{1}{2}$	7	$2\frac{1}{2}$	8
Wrought-iron	5	4	4	4
Mild steel	$7\frac{1}{2}$	6	5	10
Cast steel	5	10	$7\frac{1}{2}$	15

16. In addition to the foregoing provisions and the general rules of construction for buildings of the class to which they belong, as required by the principal Act and any amendment, all skeleton frame buildings shall, as regards their metal framing, bracing, walls, partitions, floors, roofs, staircases, and foundations, be constructed in such manner as may be approved by the district surveyor.

17. The person proposing to erect a skeleton frame building shall, one month before commencement of the building, deposit with the district surveyor a complete set of the drawings of such building showing the details of construction of all its parts, together with a detailed copy of all the calculations of the stresses and material, such calculations to be in such form as the Tribunal of Appeal shall from time to time determine. Should such drawings or calculations be in the opinion of the district surveyor not in sufficient detail, he may require such further particulars as may be necessary.

18. The district surveyor may, for the purpose of due supervision of the building and at the expense of the owner of the building, cause any pillar to be drilled at any point to ascertain its thickness, and may cause to be made any other tests he may consider desirable.

19. Any person dissatisfied with any requirement of the district surveyor under this section may appeal to the Tribunal of Appeal.

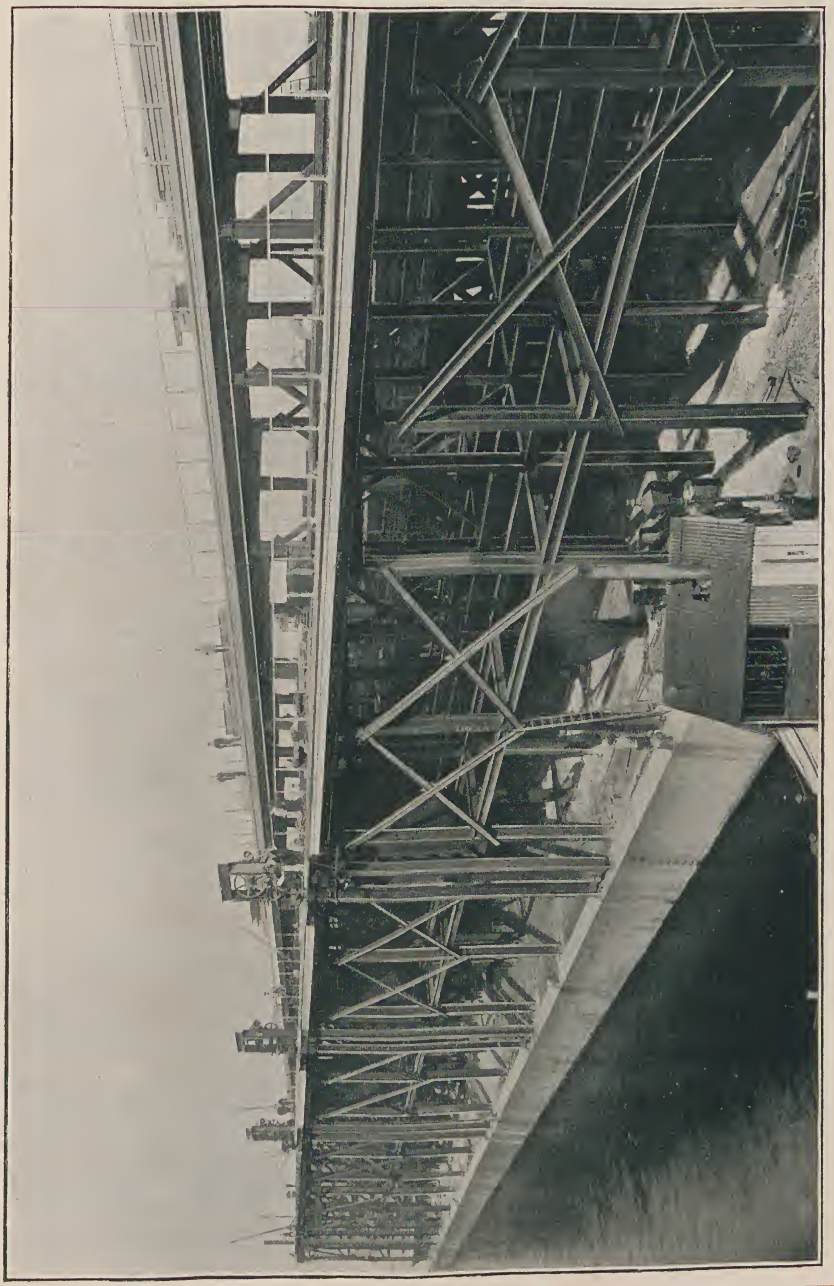
20. There shall be paid to the district surveyor by the builder or the owner in respect of every skeleton frame building, at such time as the drawings are deposited with the district surveyor, a calculation fee: such fee shall be in addition to the fee payable under section 154 of the principal Act, and shall be according to the following schedule.

COAL STAITH CONSTRUCTED OF BROAD FLANGE BEAMS.

(RIVER WEAR COMMISSION, SUNDERLAND.)

The above photograph represents a coal staith recently constructed for the River Wear Commission at Sunderland by Messrs. John Abbot & Co., Ltd., to the designs of Henry Hay Wake, Esq., M.Inst.C.E. Some interesting and valuable innovations are embodied in this new staith. The details of construction are shown more clearly in the second photograph on the opposite page. The staith is designed to carry 35-ton and 50-ton trucks and was completed in nine months. With the exception of the channel-bracing etc., the structure is composed exclusively of Broad Flange Beams. The sections used were 18″ × 12″ (496 tons), 12″ × 12″ (147 tons), 30″ × 12″ (26 tons) and 15″ × 12″ (56 tons). This is an interesting example of the scientific combination of wide-flanged sections, the structure as a whole being lighter, more rigid throughout, and better and more economical in every respect, than any possible combination of narrow-flanged material designed to carry the same loads.

COAL STAITH CONSTRUCTED OF BROAD FLANGE BEAMS.
(RIVER WEAR COMMISSION, SUNDERLAND.)



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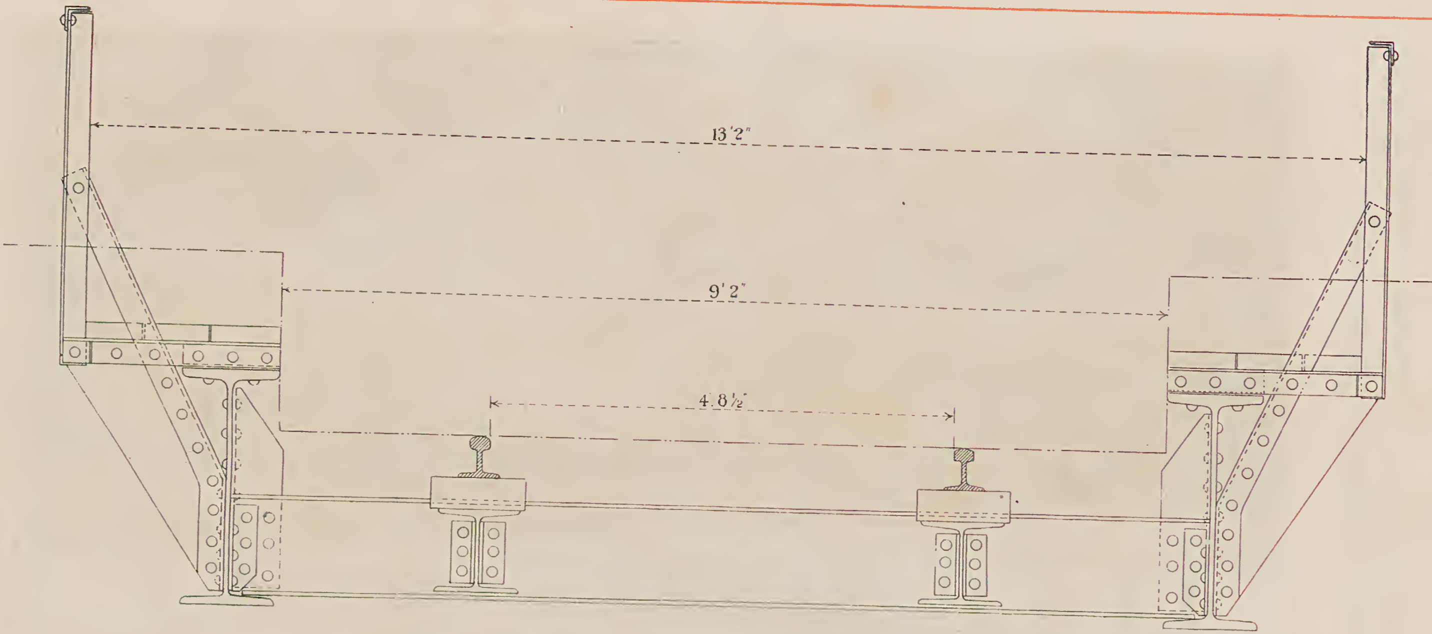


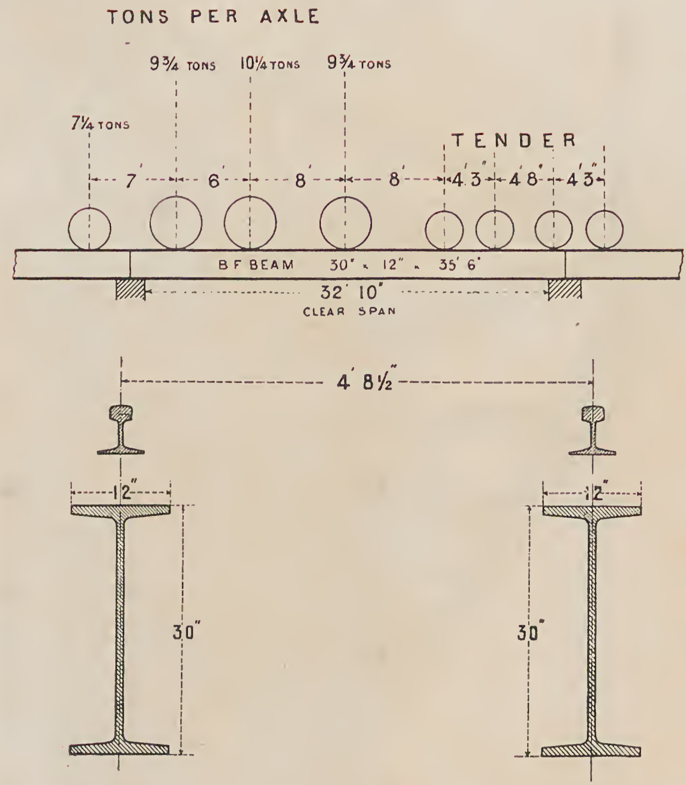
BROAD FLANGE BEAMS IN RAILWAY BRIDGES.

The above is a cross section of a railway bridge, with a single line of rails, of 27 feet span. The distance between bottom of main girder and bottom of rail is only 16 inches. The live load is about 3 tons per lineal foot, or 20 tons on an axle. The main girders are B.F. Beams, section $30'' \times 12''$, the cross girders $12\frac{1}{2}'' \times 12''$, and the rail bearers $10\frac{1}{4}'' \times 10\frac{1}{4}''$. The cross girders are 9 feet apart, and the wind-bracing is of flat bars riveted to the bottom flanges of the main girders. The railing standards are like the cross girders, 9 feet apart. By riveting flange plates to the main girders, the span of this type of bridge can be increased to 50 or 60 feet.

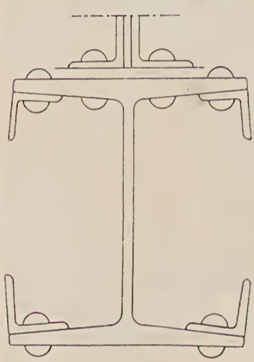
BROAD FLANGE BEAMS IN RAILWAY BRIDGE CONSTRUCTION.

DIAGRAM SHOWING BROAD FLANGE BEAMS AS USED OVER FLOOD-OPENINGS

By the Entre Rios Railway Co., Argentine Republic.
(Engineer: J. R. GARROD, Esq.)

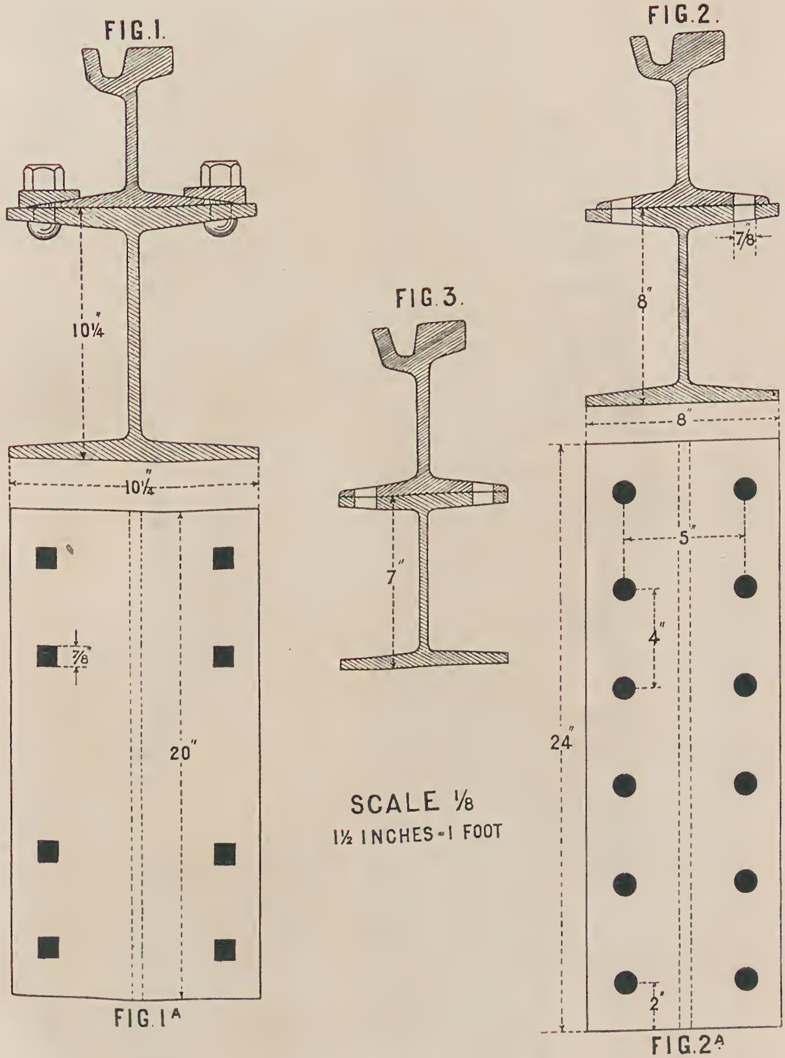


TRIANGULATED MAIN GIRDERS FOR BRIDGES.



Broad Flange Beams can be used with advantage as posts in triangulated main girders for bridges; they require either no intermediate supports in their length, or at most fewer supports than narrow-flanged joists of the same depth. A 12″ × 6″ standard joist would require three intermediate supports to make its resistance equal about both axes, while a 12″ × 12″ beam requires only one. Other advantages of Broad Flange Beams when used as main girder posts are that there is always sufficient room for riveting the cross girders of the bridge to their flanges, and also for riveting flat bars or angle bars to the inside of their flanges as a means of increasing the sectional area of the posts towards the ends of the girder, as shown in the sketch.

BROAD FLANGE BEAMS AS SOLEPLATES FOR TRAMRAIL JOINTS.

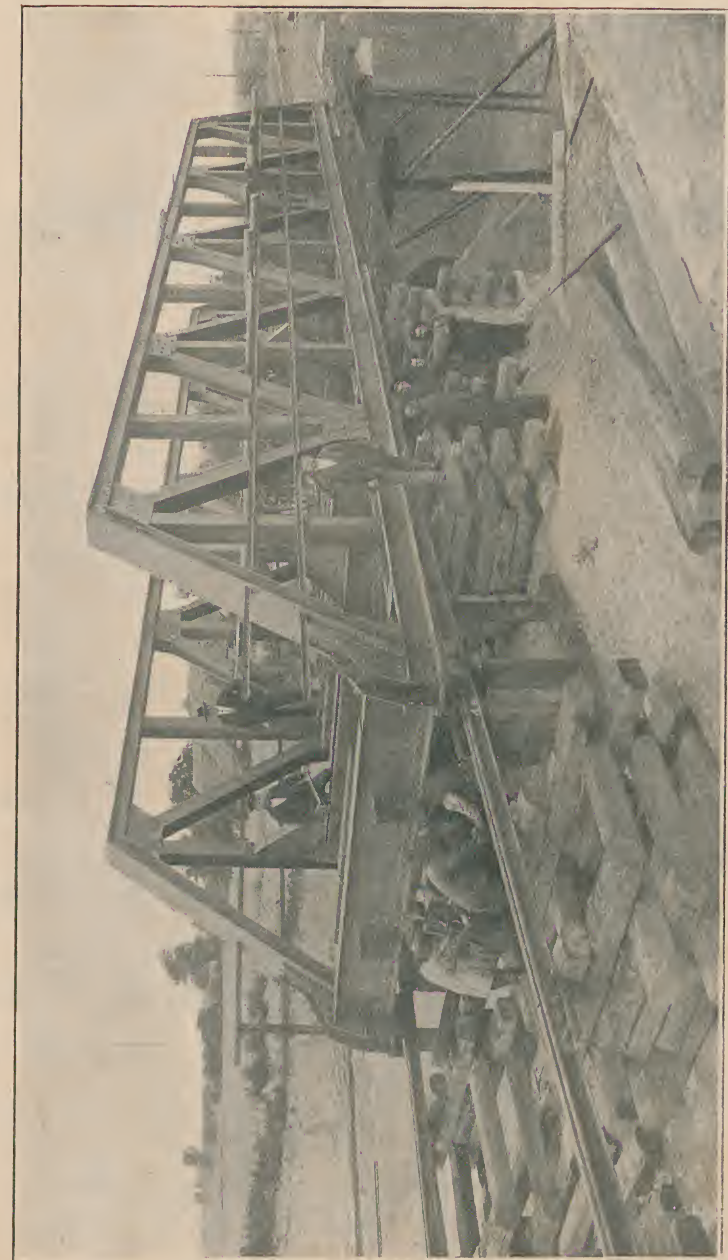


Figs. 1 and 1A show a system suggested by the United Tramways Co. as a means of saving time and expense in renewing rails. The expense of drilling holes through the flanges of the rails is also avoided.

Figs. 2 and 2A show a system adopted by the South Shields Borough Council.

Fig. 3 shows a similar arrangement using $7'' \times 7''$ instead of $8'' \times 8''$ beams.

BROAD FLANGE BEAMS IN RAILWAY BRIDGE CONSTRUCTION.



Has Holliday
Balmain